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Evaluation of the Hualien Quarter Scale Model Seismic Experiment

Description of Experiment and Summary of Results

City College of New York

U.S. Nuclear Regulatory Commission
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ABSTRACT

This report is the first volume of a four volume set describing the work performed in evaluating the results obtained from the Hualien quarter scale model seismic experiments. The experiment is described and the results of the CCNY studies are discussed in this volume.

The volumes contained in this report ("Evaluation of the Hualien Quarter Scale Model Seismic Experiment") are:

Vol. 1	Description of Experiment and Summary of Results
Vol. 2	Geotechnical Site Characterization Review
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ACKNOWLEDGEMENT

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A special acknowledgement is given to the NRC Project Manager Mr. Herman Graves. He actively participated in all aspects of the program. Recognition is also due to Dr. H.Tang of the Electric Power Research Institute who is directing the consortium of researchers conducting the Hualien project.

1.0 Introduction

This report is submitted on Contract No. NRC-04-92-049, "Hualien SSI Experiment," and this is the first of four volumes comprising the final report on the program. A listing of the title for each of the volumes is given in the Abstract. The experiment is described in this volume and the conclusions developed from the data collected at the site and the correlation of these data with predictions are summarized in this volume of the report.

A soil structure interaction (SSI) experiment is being conducted in Hualien, Taiwan. A quarter scale model reactor containment building model has been constructed at a seismically active site in Hualien. The structure and free field are instrumented so that response data within the structure and in the free field can be obtained for seismic events occurring at the site. Forced vibration tests (FVT) have been performed to evaluate vibration characteristics of the combined soil-structure system. Two such tests were performed, one before the backfill was placed and the second after the backfill was placed. These tests are discussed in the third volume to this report. instrumentation was left in place after the FVT and data was collected during seismic events which occurred at the site. These data and correlation of the measured data with predictions are discussed in volume 4. This experiment is similar to the recently completed experiment at Lotung (Ref. 1). The soil at the Lotung site was rather soft (having a shear wave velocity of about 350 fps) while the Hualien soil is relatively stiff (having a shear wave velocity of about 1000 fps). The objective of the City College of New York (CCNY) contract is to provide support to NRC in planning the experiment and in evaluating the results. Applied Research Associates (ARA) is a subcontractor to CCNY on the project. ARA had primary responsibility for the preparation of volume 2, "Geotechnical Site Characterization Review."

A description of the experiment is given in Section 2. Models used for the correlation studies are summarized in Section 3. The results of the studies are presented in Section 4. An overall discussion of lessons learned on the program is given in Section 5.

2.0 Description of the Experiment

An overview of the experiment is first given. This is followed with descriptions of the site, the model, and the instrumentation.

2.1 Overview of Experiment

The Hualien experiment is designed to develop data describing the response of a structure, similar to a nuclear power plant containment building, to seismic loadings. Particular attention is focused on the SSI process with a major objective being to correlate predictions of the model response using current SSI models with the measured responses. Overall support and direction of this project lies with an international consortium consisting of:

France:	Electricite de France (EDF)
	Commissariat A L'Energie Atominque of France (CEA)
	Framatome
Japan:	Central Research Institute of Electric Power Industry (CRIEPI)
	Tokyo Electric Power Company (TEPCO)
Korea:	Korea Electric Power Company (KEPCO)
	Korea Institute of Nuclear Safety (KINS)
	Power Engineering Company of Korea (KOPEC)
Taiwan:	Taiwan Power Company (Taipower)
United States :	Electric Power Research Institute (EPRI)
	U.S. Nuclear Regulatory Commission (USNRC)

Of course many other companies and universities have supported these groups during the project. Overall coordination of the program is provided by Dr. H. Tang of EPRI.

A "quarter scale" model containment structure was constructed in Hualien which is located in a seismically active region of Taiwan. The model was completed in 1992. Forced vibration tests (FVT) were performed to characterize the dynamic properties of the model. Two FVT were performed, one before placement of the backfill and the second after backfill. Each of the test series consisted of five parts varying the location and direction of the applied shaker force as follows: roof in the N-S direction; roof in the E-W direction; basemat in the N-S direction; basemat in the E-W direction; and basemat in the vertical direction. These tests were completed early in 1993.

Modifications were then made to the instrumentation and recording system, and the model prepared to record responses during earthquake events. The model was placed in this mode in

September 1993 and will record seismic events through 1999 at least.

A water filled surface tank has also been constructed at the site. The tank was completed in October 1994. Since useful data has not been obtained for the tank at the time of the CCNY analyses, it is not discussed in this report.

2.2 Site Description

The Hualien site is located along the east coast of Taiwan about 90 miles south of Taipei. It is located close to the coastal range fault on the milun terrace. Preliminary characterization of the site was performed by the Institute of Earth Sciences (IES) and United Geotech Inc. These studies indicate that the surface material consists of two layers. The top layer is shallow (about 18 ft) and consists of a silty sand with a small gravel content. This material has a shear wave velocity about 440 fps. The second layer is a gravely sand having shear wave velocities around 1315 fps and this layer ranges in depth from 330-660 ft. The gravel in this layer ranges in size from 1-1/4" to 2-3/4". The underlying material is a mudstone with a shear wave velocity equal to about 4900 fps.

The final geophysical characterization of the site was done by CRIEPI. The data from these and the preliminary studies are discussed in detail in volume 2. These include descriptions of the soils exploration program and the resulting soil properties. CRIEPI developed "unified soil models" to be used is predicting the model responses. These soil properties are shown on Figure 2.1 together with a sketch of the containment model. It is interesting to note that the in situ near surface soil is significantly softer than the backfill material.

2.3 Structural Properties

The characteristics of the quarter scale containment structure model are discussed in this section of the report. A sketch showing the primary dimensions of the model are shown on Figure 2.1. Openings in the model are negligibly small. The model is constructed of concrete having a compressive strength of 5 ksi and an elastic modulus of about 4,088 ksi. Poisson's ratio is taken to be equal to 0.2.

Eigenvalue solutions were obtained by modeling the containment as a fixed base shell and as a shear beam. Fundamental vibration frequencies were computed using the ABAQUS computer code (Ref. 2). The first two modes for the shell model were found to be 10.4 cps and 31.7 cps. A review of the mode shape information indicated that circular cross sections remained circular. For example, the relative displacement, in one of the horizontal directions, of a circle located at midheight varied from 0.313 to 0.347. If these displacements were identical the circular cross section would remain circular during vibration. The first two mode shape frequencies obtained from the beam model were found to be 10.0 cps and 29.8 cps. Since the beam model frequencies are within 10% of the shell model frequency the containment is modeled as a beam structure.

The mass data for the model is shown on Table 2.1. The locations of the nodes indicated on Table 2.1 are given in the next section of the report dealing with modeling. The mass data result in a total weight of 3,133 kips, a center of gravity located 25.87 feet above the bottom of the basemat, and a rotary inertia about the base of 116,300 k-ft-sec^2. The bearing capacity under the containment is about 3.3 ksf and the containment structure weighs slightly less than twice the weight of the soil it displaces. These data bear on the SSI characteristics of the model.

Table 2.1

Mass Properties of the Model

Node	Distance Above Base (ft)	Weight (kips)	Rotary Inertia (kip-ft-sec^2)
1	0.00	730.45	1766.7
2	9.84	781.22	2187.6
3	16.40	99.23	822.6
4	22.66	122.37	803.3
5	32.21	129.26	1071.6
6	39.36	120.67	1224.3
7	47.80	607.53	2618.1
8	52.92	542.20	2063.7

The 0.98 feet thick cylindrical shell has a cross sectional area of 103.65 square feet, a shear area of 51.83 square feet, and a moment of inertia of 14,750 ft^4. Significant structural stresses are not expected so that structural damping is taken at 2 % of critical.

Grade 60 reinforcement is used in the model, but the expected stresses are not large enough to crack the model. Uncracked properties are used for the study therefore, and the reinforcement properties do not influence the modeling.

2.4 Instrumentation

Instrumentation for the FVT and for the earthquake measurements are discussed in this section of the report.

2.4.1 Forced Vibration Test Measurements

The forced vibration test series were conducted by Kajima both before and after placement of the backfill. Five tests were conducted in each series. Two tests were conducted with the shaker located on the roof with the excitation in the N-S and E-W directions. The other three tests were conducted with the shaker located on the basemat with the excitation in the N-S, E-W, and vertical directions. It should be noted that the "north" direction is defined as "plant north" for the FVT which is 41.25^{o} west of magnetic north. The shaker delivered peak forces ranging from about 100 kg at 2 cps to about 9,000 kg at 20 cps.A sinusoidal shaped forcing function was used with a frequency range from 2 cps to 20 cps for the horizontal tests and from 2 cps to 25 cps for the vertical tests. Data was collected at 0.2 cps increments except near the fundamental frequency where frequency increments of 0.1 cps were used.

The instrumentation consisted of velocity type displacement meters and pressure cells. The pressure cells were placed to monitor wall/soil and basement/soil interface pressures during earthquakes. Useful data was not obtained for the pressure cells during the FVT. As a result they are not discussed further. The displacement meters were placed in the model and in the free field. The placement in the model was on the roof, at midheight, and on the basemat The gages were generally placed around the model periphery at each of the elevations and recorded responses in the N-S, E-W, and vertical directions. Locations of the gages for each of the tests are shown on Table 2.2. The nodal locations are discussed in Section 3 and defined on Figure 3.1

Displacement meters are placed in the free field along radial lines north and east of the model. The three gages along each radial line are located 4.9 feet, 19.7 feet, and 31.2 feet out from the wall of the model. The furthest gages are located on the surface of the soil and outside of the backfill area. The two closest gages on each of the radial lines are located on the ledges of the excavated zone (see Figure 2.1) for the before backfill tests and on the surface for the after backfill tests.

Table 2.2

Locations / Directions of In-Structure Measurements

Node	Test				
	Roof N-S	Roof E-W	1st Flr N-S	1st Flr E-W	1st Flr U-D
2	UD	UD	UD	UD	UD
8	NS, UD	EW,UD			UD
12	EW, UD	EW, UD	EW, UD	EW, UD	EW, UD
13	EW, UD	EW, UD	EW, UD	EW, UD	EW, UD
14	NS,UD	NS, UD	NS, UD	NS, UD	NS, UD
16	NS, UD	NS, UD	NS, UD	NS, UD	NS, UD
17	NS	NS	NS	EW	
18	UD	EW	UD	*(NS)	UD*( )
19	EW	EW	EW	EW, UD	*(UD)
20	NS	UD	NS	NS*( )	
21	EW, UD	EW, UD	EW, UD	EW, UD	EW, UD
22	UD		UD		
24	UD		UD		
25	EW, UD	EW, UD	EW, UD	EW, UD	EW, UD
26	NS, UD	NS, UD	NS, UD	NS, UD	UD*(NS,UD)
27		UD			
29		UD		*(UD)	
30	NS, UD	NS, UD	NS, UD	NS, UD	NS, UD

* FVT-1 and 2 locations are different. FVT-1 location given before asterisk and FVT-2 location is given after asterisk in brackets.

Analog signals are obtained from both the shaker and the displacement meters. These signals are digitized and fed into a personal computer. A cross correlation analysis is performed on the displacement meter signal to minimize the noise, and then this signal is used with the shaker force input to obtain amplification and phase angle results. The amplification results are reduced to a common scale of micro meters / ton. The data was supplied in this format to members of the consortium.

2.4.2 Earthquake Instrumentation

The instrumentation placed after the FVT and remaining in place to record seismic events consist of accelerometers, pressure gages, pore water pressure gages and settlement gages. The location of each of these are given and the format of the raw data is discussed.

Accelerometers are located on the surface of the free field along three radial lines as shown on Figure 2.2. The radial lines are separated by about 120^o with Arm 3 oriented 68.48^o east from magnetic north. There are five stations along each of the radial lines located 0.5, 1.0, 2, 3.5, and 5.5 diameters from the center of the model. The accelerometers are designated a_{ij} with i indicating the arm and j indicating the location on the arm. The j index ranges from 1 through 5 with a_{i1} being located on arm i and closest to the model. Each gage records N-S (magnetic), E-W, and vertical accelerations. All of the records are generally about 40 seconds in duration. The a_{ij} gages give the acceleration in terms of "counts." The gage factor for the gages is 16384 counts/g except for the a_{i1} gages where the factor is 18750.

Accelerometers are also located in three downholes located directly under the surface gages a_{15}, a_{21}, and a_{25}. Each hole contains four gages located at depths of 17.33 feet, 51.84 feet, 86.29 feet, and 170.6 feet below the surface. The gages under a_{15} are numbered d_{11}, d_{12}, d_{13}, and d_{14} from the top to bottom of the hole. The gages under a_{21} are numbered d_{21}, d_{22}, d_{23}, and d_{24} from the top to bottom of the hole. The gages under a_{25} are numbered d_{25}, d_{26}, d_{27}, and d_{28} from the top to bottom of the hole.These gages also record accelerations in the N-S, E-W, and vertical directions. The gage factor to convert counts to g's is 16384 counts/g for all of the buried accelerometers.

Accelerometers are located at three elevations in the structural model: on the basemat, 6.56 feet above the basemat (at the elevation of the ground surface), 22.37 feet above the basemat (midheight of the model), and at the top of the roof (43.08 feet above the basemat). The locations of these gages are given on Table 2.3. The "X" coordinate is in the east direction and the "Y" coordinate is in the north direction. All of the records are generally about 40 seconds in duration. The gages give the acceleration in terms of "counts." The gage factor for the structural model gages is 18750 counts/g.

The raw accelerometer data were received on floppy disks together with an Institute of Earth Sciences (IES) report in which the data was converted to g units and baseline corrected. Time plots were given of the records, response spectra of the record, and Fourier amplitudes. The raw records were processed at CCNY in the following manner to develop the accelerograms used in the analyses described in Volume 4: the records were converted from counts to g's; the Arias intensity functions were generated for selected records; the plots contained in the IES reports and the Arias intensity data were viewed to select the time window to be analyzed (usually between 20 and 30 seconds); all of the records were cropped to this time frame; and the records were baseline corrected (a parabolic function was fit to the accelerogram and removed).

Table 2.3

Locations of Seismic Accelerometers on Model

Gage Designation	Elevation	X (ft)	Y (ft)
BAN	Basemat	0.	15.58
BAE	Basemat	15.58	0.
BAS	Basemat	0.	-15.58
BAW	Basemat	-15.58	0.
WLN	Gnd. Elev.	0.	15.18
WLE	Gnd. Elev.	15.58	0.
WHN	Midheight	0.	15.58
WHE	Midheight	15.58	0.
WHS	Midheight	0.	-15.58
WHW	Midheight	-15.58	0.
RFN	Roof	0.	15.76
RFE	Roof	15.76	0.
RFS	Roof	0.	-15.76
RFW	Roof	-15.76	0.

Pressure gages are located along the foundation soil interface as shown on Figure 2.3. The pressure gages record pressure in counts which can be converted to pressure in units of kg/cm^2 by dividing the counts by 3550 for the gages PR 1 through PR 9 and 10650 for the remaining gages. Plots of the pressure time histories are included in the IES reports.

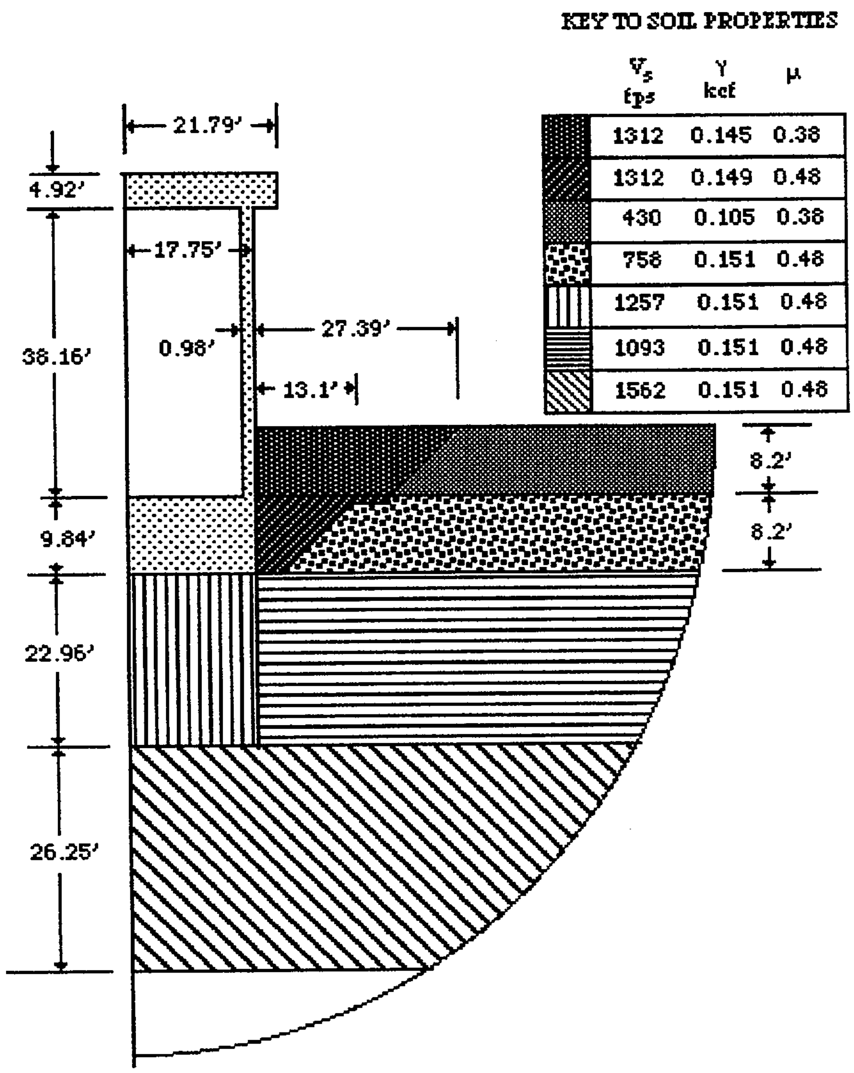


Fig. 2.1 CRIEPI Unified Soil Model and Containment Model Dimensions

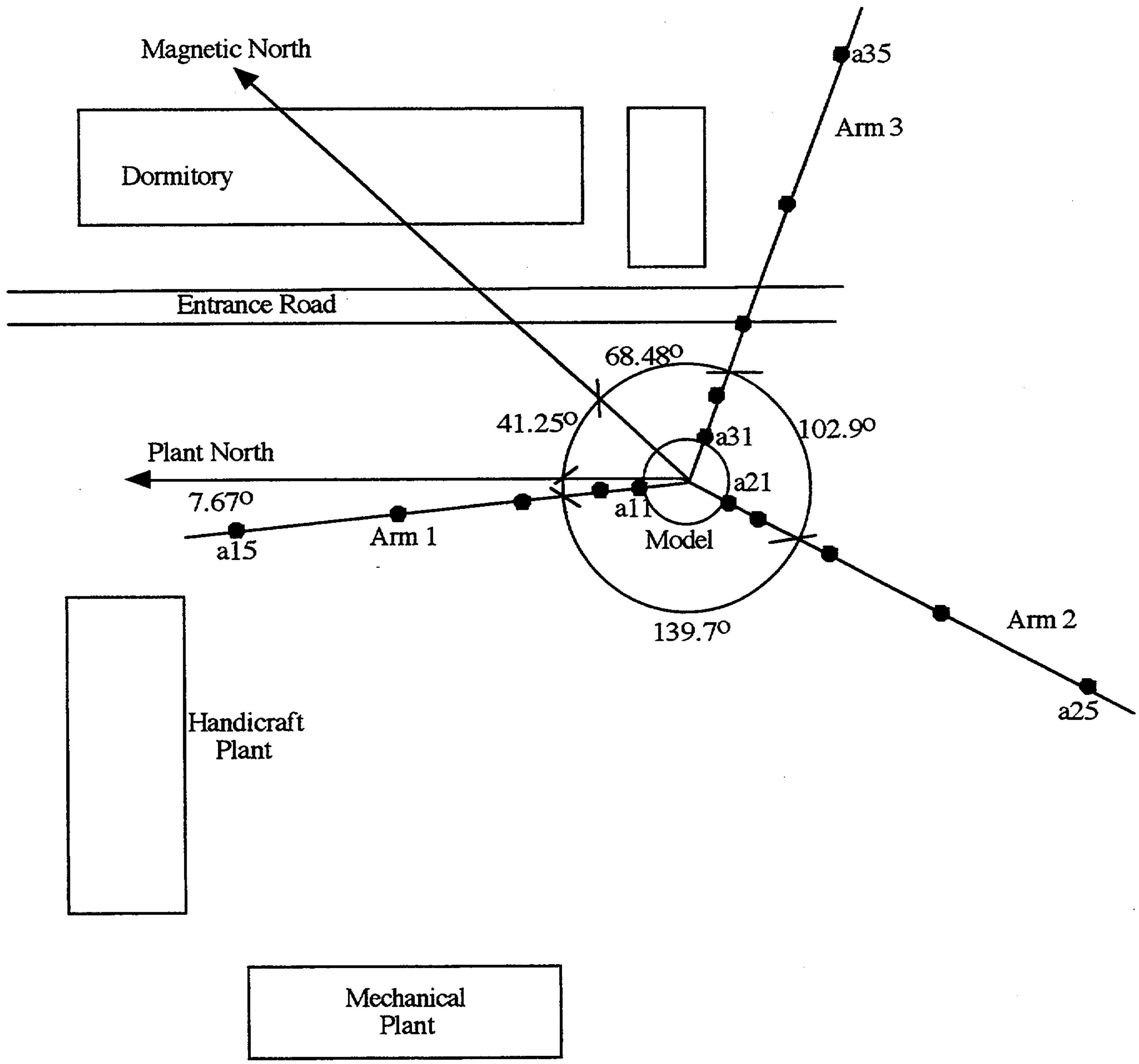
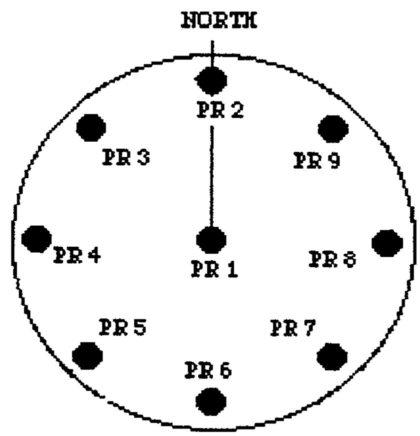
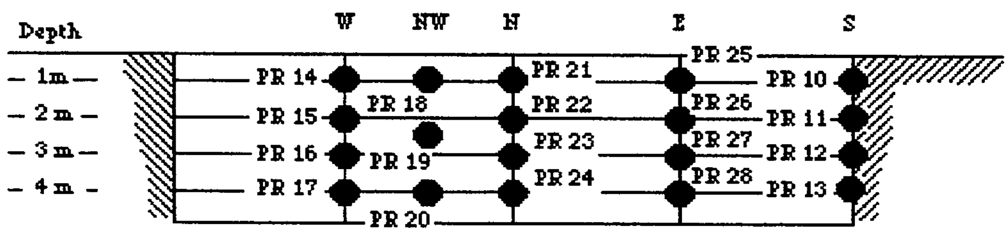


Fig. 2.2 Plan of Surface Accelerometers



(a) On Basemat / Soil Interface



(b) On Expanded View of Wall / Soil Interface

Fig. 2.3 Pressure Gage Locations

3.0 Models Used For Study

The CARES computer program (Ref. 3) is used for all of the correlation studies performed at CCNY. This program is discussed first. The structural models and soil models used for the correlation studies are then described.

3.1 CARES Program

The CARES (**Computer** Analysis for the **R**apid **E**valuation of **S**tructures) computer program (Ref. 3) was developed for the Nuclear Regulatory Commission to provide a relatively easy method for analyzing seismic effects on structures. The program contains three subprogram modules: a soils module; a structural module; and a service type module. Each of these are briefly described.

The soils module performs two primary tasks. In the first task an accelerogram can be generated which fits a prescribed response spectra. The computations begin with an accelerogram which may be a white noise pulse or an accelerogram specified by the user. There is no need to perform this task for the Hualien study since all of the ground motions are based on measured data. In the second task free field motions are calculated based on a vertically propagating horizontally polarized shear wave model. This module is used for the free field motion correlation studies discussed in Volume 4. The motion at various depths in the free field are calculated based on a given motion at one depth and the soil properties (shear modulus, shear modulus degradation with shear strain, unit weight, Poisson's ratio, and soil damping which can increase with increasing shear strain). The soil column is divided into layers with properties specified for each layer. The solution is then carried out in an iterative manner: solution found for the specified soil properties, soil shear strains evaluated in each layer (effective shear strain is taken to be 65 % of the maximum shear strain in a layer), degraded soil properties determined, and the process repeated until the soil properties converge. Several soil property degradation models are built into the code or the user may specify the degradation model (which is done for this study).

The structural module determines the response of the structural model to a time dependent disturbance. The disturbance can be either a harmonic forcing function (used for the forced vibration test correlation studies discussed in Volume 3) or a ground motion (used for the earthquake response correlation studies discussed in Volume 4). The structural model is linear elastic and may be made up using 3-D shear beams, plane shear wall elements, and lumped masses. Rigid connections between nodes in the model may also be specified. Soil - structure interaction effects are modeled with frequency dependent impedance functions connecting the foundation of the structure to the ground. The ground motion is specified for the earthquake application. Several options are available for the impedance functions. They are numerically application. Several options are available for the impedance functions. They are numerically defined in the code for the cases of either a circular or rectangular foundation embedded in the soil. The soil is restricted to a two horizontal layer system: one beneath the foundation and one to the side of the embedded structure. The impedance functions contained in the code are obtained from: ASCE 4-86 (Ref. 4), Beredugo & Novak (Ref. 5), or Kausel (Ref. 6).

The service module contains general applications that are necessary to process the data. The free field and structural computations are carried out in the frequency domain. A FFT capability is included in the code to convert accelerograms to Fourier transforms and to convert back to time histories. Response spectra can be generated for any given history. These are used to process the data computed during the soils and structures modules.

3.2 Structural Model

The containment building (see Figure 2.1) is modeled with 3-D shear beams and rigid links as shown on Figure 3.1. A shear beam is used to model the cylindrical shell portion of the structure and it spans from nodes 2 through 7. Rigid links connecting nodes 7-8 and 1-2 are used to model the roof and basemat slabs both of which are considered to be rigid. Nodes 9 and 10 are included in the model to represent the cg of the shaker on the roof and first floor respectively. These nodes are connected with rigid links to nodes 8 and 2. The remaining nodes (11 through 30) are used to replicate accelerometer locations in the structure. All of these nodes are connected with rigid links to the nodes 2 through 8 on the structural model. The "north" arrow on Fig. 2.2 corresponds to the "north" definition used in the tests. When the model is used for the forced vibration test predictions north is about 41.25^{o} west of magnetic north at the site. When the model is used for the earthquake predictions north corresponds to north at the site. The fixed base fundamental frequency of the structure is about 10.7 cps. Stresses in the structure are low for all of the responses (both from the FVT and the earthquakes), and structural damping equal to 2 % is used for all predictions.

Node 1 (located at the center of the bottom of the basemat) is attached to the free field through frequency dependent Beredugo and Novak (Ref. 5) impedance functions. The fundamental frequencies of the combined soil / structure system using the low strain soil properties to evaluate the impedance functions are found to be 4.3 cps and 6.0 cps for the before and after backfill cases respectively. These modes are associated primarily with rigid body rocking of the structure. The roof deformations are divided into 55 % rocking deformation, 40 % elastic structural deformation, and 5 % rigid body translation for the after backfill cases. Of course the after backfill case is applicable for the seismic predictions. The fundamental frequency of the system for the seismic predictions is somewhat lower than 6 cps since the soil strains during the seismic events result in

3.3 Soils Model

The soil column model used to perform the convolution studies for the seismic events is shown on Figure 3.2. This soil column is located under the surface gage a25 and is at the end of arm 2 (see Figure 2.2) and therefore outside of the backfill area. The layering as defined by the CRIEPI "unified soil model" is shown shaded on Figure 3.2. Each of these layers is subdivided as shown so that realistic estimates of peak soil strains may be determined. Soil property and degradation models are shown on the figure. Maximum shear strains for the earthquakes evaluated are less than 0.007 % with average values closer to 0.003 %. These are quite small so that degradation effects do not play a significant role in the predicted responses.

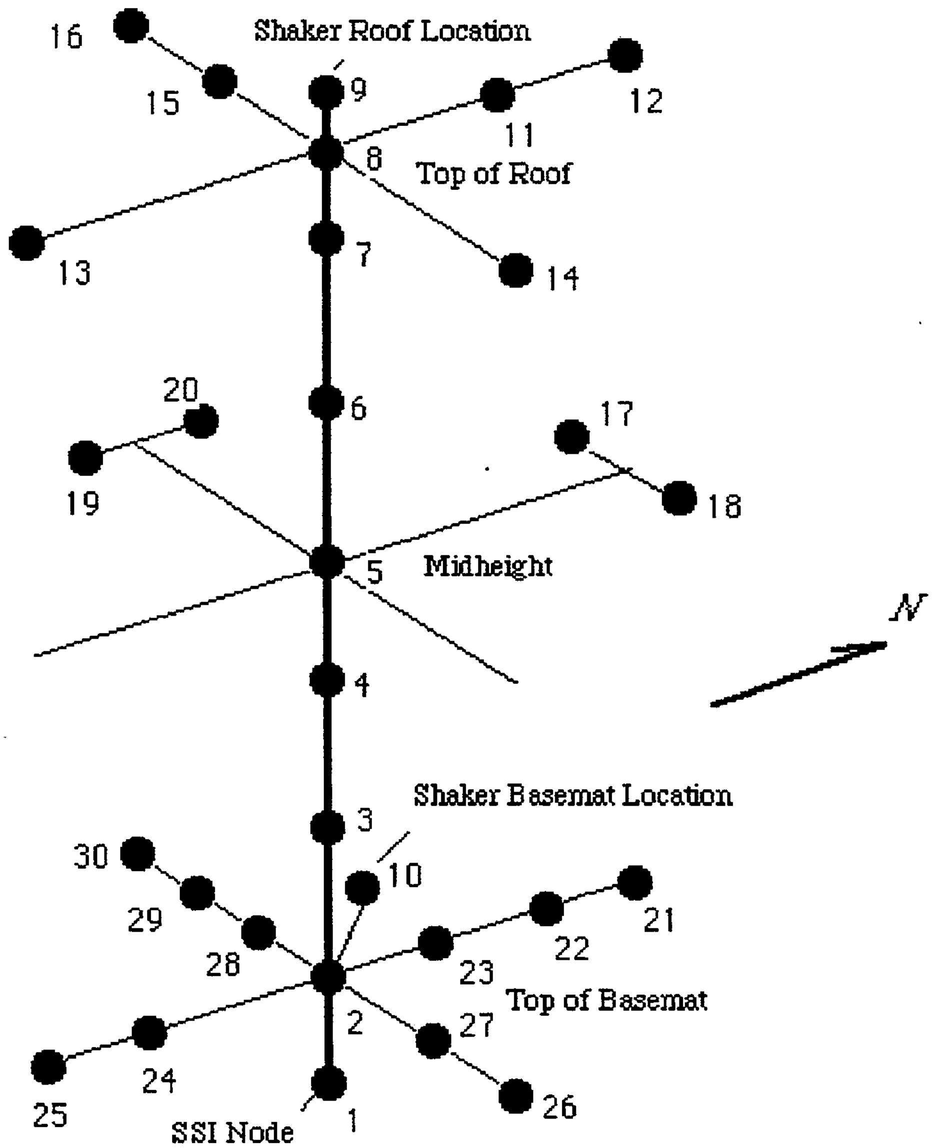
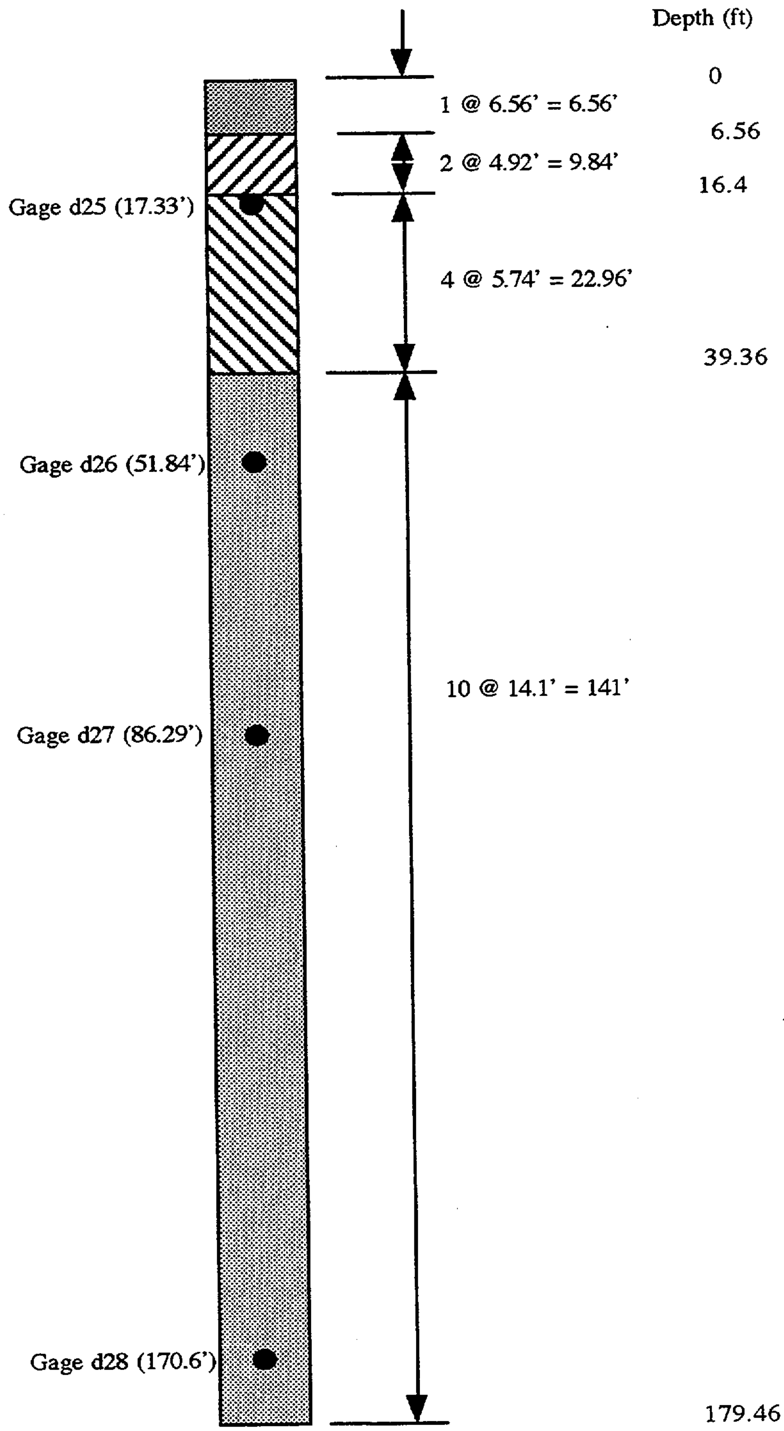


Fig. 3.1 Description of Structural Model



Shear Vel (fps)	Density (pcf)
436	105
758	120
1093	151
1562	151

SOIL DEGRADATION MODEL

Strain (%)	G/Go	Damping (%)
.0005	1.000	2.000
.0008	0.980	2.100
.0015	0.940	2.300
.0039	0.830	3.200
.0060	0.780	3.900
.0095	0.700	5.100
.0200	0.570	7.500
.0380	0.390	11.400
.0570	0.290	14.300

Fig. 3.2 Soil Column Model Used for Convolution Analyses

4.0 Summary of Results

The results of this project are discussed in detail in the following three volumes dealing with: the soils exploration; the forced vibration tests; and the seismic response studies. The material presented in each of these volumes is summarized in this section.

4.1 Geotechnical Site Characterization Review (Volume 2)

The site characterization work was done by CRIEPI. The CRIEPI work extended from the onset of the project to the completion of the project. The CRIPEI results are contained in a series of reports submitted during the project and the definition of "unified soil models" giving properties to be used by the consortium members for their predictions. The CRIEPI work is reviewed and summarized with the objective of developing a comprehensive geotechnical report. The following conclusions are drawn from this review:

1. A comprehensive site characterization program was conducted by CRIEPI between 1990 and 1994. This site characterization program paid particular attention to the soil conditions to a depth of about 12 m below the ground surface (GL-12m). In this regard, the site characterization was at least as thorough as those commonly used for actual, prototypical, sites. In particular, a detailed characterization was made for the surface layer of sand from GL0 to GL-5m, for the gravel beneath the foundation and in the free-field from GL-5m to GL-12m, and for the as-built backfill soil. A somewhat less detailed characterization was provided for the gravel between GL-12m and GL-20m. The site characterization for depths below about GL-20m appears to have been beyond the scope of the effort by CRIEPI and, at present, it seems to rely almost exclusively on the scoping geophysical study performed in the early stages of the Hualien SSI program by the Institute of Earth Sciences (IES) of the Academia Sinica. This characterization may be limited for the purposes of detailed correlation analysis studies with the ground accelerations recorded in the down-hole arrays, which extended to a depth of about GL-52.5m. Examples of such correlation studies are: site amplification studies and deconvolution analysis.

2. An important feature of the site characterization was that it paid particular attention to the effect of the various construction operations on the in-situ shear wave speed and soil stiffness. This was shown by CRIEPI to be important for the cohesionless soils encountered whose stiffness and strength depend on the effective stress conditions at the site.

3. Shear wave speeds measured with down-hole, cross-hole and suspension logging are consistently greater than those inferred from laboratory tests on samples collected by ground

freezing techniques. The reasons for such discrepancies are not known at present. These discrepancies have been attributed to possible sample disturbance resulting from high fine contents of the order of 15 to 25%. They may be the result of numerous pebbles or cobbles as large as 10 to 20 cm in diameter that would require testing specimens greater than those used, or a result of the difficulty to obtain in-situ shear wave speed measurements for a gravel with numerous pebbles or cobbles as large as 10 to 20 cm in diameter. They may also be the result of soil anisotropy as discussed below.

4. In-situ saturated densities recommended by CRIEPI for the as-built backfill soil and the gravel to a depth of about 20 m below ground level are about 2,390 kg/m3 (a dry density of 2,200 kg/m3) and 2,420 kg/m3, respectively. These are unusually high densities which are similar to those of concrete. The gravel is a well-graded soil for which coefficients of uniformity of the order of 150 to 400 were computed. Such a gradation may lead to a very low void ratio which would explain the high bulk density of this soil. The backfill soil is not well-graded, and its reported dry density of about 2,200 kg/m3, which is based on the quality control tests conducted for the backfill compaction, is unusually high for a compacted soil. It is also noted that the dry densities for the reconstituted backfill material used to establish the variation of the secant shear modulus and damping ratio with shear strain are about 10% less than that recommended for the in-situ conditions for FVT-2 analysis.

5. A Poisson's ratio of 0.48 is recommended for the backfill soil between GL-2m to GL-4m for the FVT-2 analysis. The Poisson's ratio for this soil computed from measured S and P wave speeds is 0.24. Full saturation must be assumed for a Poisson's ratio of 0.48. It is improbable that the backfill soil from GL-2m to GL-4m would have been fully-saturated at the time of the FVT-2 test. Full saturation is more likely in early 1994 when significant earthquake loads were recorded at the site. Nevertheless, full saturation will depend on whether the depth of the groundwater table at the site remains approximately equal or changes significantly from year to year and/or throughout the year.

6. Cross-hole logging results reveal S wave speeds from GL-5m to about GL-12m that appear to be significantly different in two approximately orthogonal directions both before excavation and after construction of the model. The data, however, show considerable scatter and the difference between the wave speeds in the two orthogonal directions is more apparent in terms of the averages of the measurements in each direction. Prior to the forced vibration tests, the possibility of anisotropic site conditions with two principal directions of shear wave propagation in the horizontal direction was not expected. The forced vibration test data and the recorded earthquake ground motion data, however, reveal that this anisotropy appears to be the actual site condition.

from the laboratory testing as compared to those from in-situ shear wave speed measurements. Assuming that the soil anisotropy is preserved through the sampling process, the shear deformation of the soil under torsional and triaxial loading conditions is not likely to be representative of the soil stiffness in the strong direction. Indirect evidence of this is that the shear wave speeds in the sands measured in the laboratory are not less than those measured in the field, and the sands encountered in the upper 5 meters appear to be isotropic. It is also possible that the soil anisotropy is not preserved during sampling and that a simple consolidation of the specimen under isotropic stress conditions will not properly reconstitute the in-situ conditions. Shear strength parameters for the gravel obtained from triaxial testing are also likely to be affected by anisotropy and the triaxial test results are more likely to be representative of the soil strength in the weak direction rather than that in the strong direction, if the soil strength is anisotropic as the shear stiffness appears to be.

4.2 Results of the Forced Vibration Tests (Volume 3)

The FVT tests consisted of five shaker experiments performed on the Hualien quarter scale model. The shaker was placed on the roof with the excitation in the N-S and E-W directions for the first two tests, the third and fourth tests were conducted with the shaker placed on the basemat and the excitation applied again in the N-S and E-W directions, and the shaker excitation for the fifth test was in the vertical direction with the shaker located on the basemat. Two sets of such tests were performed, one before placement of the backfill and one after placement of the backfill.

The measured data is reviewed and response characteristics (fundamental frequency and damping) are derived from these data. The model response is developed using the structural model described in Section 3 and these predictions are compared with the measured data. The following conclusions are drawn from the study:

1. The peak responses for the horizontal shaker tests (both with the shaker at the roof and basemat levels) occur at 4.1 cps when the excitation is in the N-S direction and 4.6 cps when the excitation is in the E-W direction. The contributions to the measured peak response at the roof level for the excitation placed at the roof is about 11-12 % rigid body translation, 66-69 % rigid body rocking about the base, and 19-23 % flexural deformation of the structure.

2. The difference in frequency for the N-S and E-W response is likely due to some anisotropic behavior in the soil. The "stiffer" behavior in the E-W direction is also demonstrated by the peak measured responses. The peak response in the N-S direction due to N-S excitation is 213 μm/t and the peak response in the E-W direction due to E-W excitation is 175 μm/t. The excitation in each of the horizontal directions also results in an out of plane response that is

μm/t and the peak response in the E-W direction due to E-W excitation is 175 μm/t. The excitation in each of the horizontal directions also results in an out of plane response that is about 60 % and 35 % respectively for the N-S and E-W excitations. The soils exploration program did not uncover this anisotropic characteristic although it can be observed in the data.

3. The amplifications of the horizontal displacements indicate that the effective damping in the system is about 7 %.

4. The measured results from the vertical excitation case indicate that the system is highly damped in this direction (greater that 50 %) and that the vertical system frequency is about 11 cps.

5. The correlation studies comparing the predicted (using the unified soils model) and measured results show good agreement. The predicted fundamental frequency for the horizontal tests is 4.3 cps compared with the measured 4.1 cps and 4.6 cps. The predicted roof horizontal amplitude is 207 μm/t and compared with the measured 213 μm/t and 175 μm/t. Of course the models do not predict the out of plane response since the models are symmetric. There is no bases in the data describing the experiment to incorporate nonsymmetries.

Evaluation of the data obtained from the forced vibration tests conducted after placement of the backfill leads to the following conclusions:

1. The measured response of the structure caused by the horizontal shaker loads indicate that the system frequency is about 6.2 cps with peak response at the roof being about 62 μm/t. Comparison of these data with the FVT-1 results indicate that the backfill has increased the frequency by about 50 % and decreased the peak response by a factor of about 3 to 4. This is a modest increase in frequency and a significant reduction in response. It can be concluded that the principal effect of embedment is to increase the damping and thereby reduce the response. The embedment effects of the test structure would be expected to be more significant than found for most nuclear power plant structures since the depth of embedment is almost equal to the radius and the backfill material has a stiffness larger than the underlying material.

2. The out of plane response (N-S loading introduces E-W response) found in the FVT-1 results is also found for the FVT-2 results although the effect is much less. This would indicate that the backfill material results in responses in the same plane as the loading and thereby reduces the out of plane responses caused by the underlying material.

3. The horizontal loading introduces a small vertical rigid body deformation of the structure. This

results from rigid body rocking of the structure, 5 % results from rigid body translation of the structure, and the remaining 40 % results from flexure.

5. The predictions are in excellent agreement with the measurements for the horizontal shaker experiments. For example the predicted fundamental frequency is 6 cps as compared with the measured 6.2 cps, and the predicted amplitude of the roof displacement is 58 μm/t as compared with the measured 62 μm/t. The predicted distribution of the roof displacement between the rigid body and flexural modes is very close to the measured data,

6. A correlation study of the effect of the soil shear wave velocities indicates that good correlations between the measured and computed results would be obtained if the underlying soil shear wave velocity is between 383 m/s and 500 m/s and for side soil shear wave velocities in the range of 300 m/s and 400 m/s. The CRIEPI "unified soils models" are within these ranges.

7. The vertical shaker test resulted in a fundamental frequency of 10.8 cps and peak vertical responses of about 0.6 μm/t. This frequency is about the same as found for the FVT-1 test but surprisingly the displacements are about 60 % of those measured in the FVT-1 test. It is surprising that the backfill has a significant effect for the vertical excitation test.

8. The predicted vertical response is similar in magnitude and frequency with the measured data but the shape of the amplitude - frequency curve is quite different.

4.3 Response of the Model to Seismic Events (Volume 4)

Data recorded during four earthquakes are considered in this study. Two of the earthquakes are about magnitude 5.5 events and occurred about 22 km NE of the site. The peak recorded free field and in-structure accelerations during these events are 0.06 g's and 0.08 g's respectively. The other two events are about magnitude 4.7 events and occurred about 5 km from the site. One was located NE of the site and the other SE of the site. The peak accelerations recorded are 0.14 g's and 0.17 g's in the free field and structure respectively. One earthquake from each is selected for detailed study (the magnitude 5.8 February 23, 1995 event and the magnitude 4.9 May 1, 1995 event).

A series of studies are performed using the measured data to obtain some insight to the significant characteristics of the model. In particular these studies are performed with the objective of evaluating the extent to which the earthquake data is consistent with the results of the forced

A series of studies are performed using the measured data to obtain some insight to the significant characteristics of the model. In particular these studies are performed with the objective of evaluating the extent to which the earthquake data is consistent with the results of the forced vibration tests conducted after backfill. The forced vibration tests indicate that significant nonsymmetric responses occur (attributed to anisotropic site properties) and that the primary SSI frequency is 6 cps and associated with the rocking mode. The following is a summary of the studies and results:

1. Transfer functions are developed between the responses at the roof of the model and the surface gages in the free field. These transfer functions indicate that there is a significant coupling between the free filed horizontal input motion in the L and T directions and the model response. The measured data are used to determine principal directions which are found to be 29^o east of the L (North) direction. These principal directions are about 14^o different from the principal directions as found from the forced vibration tests and confirm the anisotropic characteristics of the site.

2. Transfer functions between each of the gages in the downhole array are developed. This is done both in the L / T measurement coordinate system and in the principal X / Y system. The Transfer functions are found to depend on the direction giving further evidence of anisotropic effects. The frequency at which the initial peaks in the the transfer functions occur are in the order: Y>L>T>X. This indicates that the site is stiffer in the Y direction than in the X direction as was also found from the forced vibration tests. The fact that the stiffness in the L / T directions falls within the stiffness in the X / Y directions is also consistent with the determination that the X / Y coordinates represent principal directions.

   It is also found that this anisotropic effect extends down to the deepest gages (between gage d27 at 86 feet and d28 at 170 feet). This supports the conclusion that the anisotropic characteristics are site wide and not restricted to a region around the model.

3. The fundamental frequencies in the soil column as found from the transfer functions support the conclusion that the soil may be softer than suggested in the CRIEPI unified soil model. In particular it appears that a soft layer in the soil column occurs somewhere between a depth of 52 feet (gage d 26) and a depth of 86 feet (gage d27).

4. Transfer functions between the roof of the model and the furthest surface free field gages (a15, a25, and a35) are found to depend on direction (X / Y or L / T) and on the free field gage used to define the control motion. The dependence on direction again reflects anisotropic site characteristics, and the dependence on location of the control point indicates that difficulty is

four gages located in the downhole. This is done in the three directions X, Y, and V. The unified soil model properties are used with the soil moduli degraded and damping increased depending on soil strain and the degradation model included with the unified soil model. Spectra of the computed motion at the four gage locations are then compared with spectra of the recorded motion. The following conclusions are drawn from these comparisons:

1. Reasonably good agreements in the horizontal components of the motion are found over the entire depth of the soil column with the Y direction correlations being somewhat better than those in the X direction.

2. The correlations between the measured and computed spectra of the horizontal motions are better at gages d25 and d28 than at gages d26 and d27. This may result from the fact that the column frequencies above gages d26 and d27 are close to principal frequencies of the input motion (1.5 - 3 cps).

3. The correlations between the measured and computed vertical spectra are good at all depths for frequencies less than 10 cps. The computed spectral accelerations are higher than for the measured data at these higher frequencies.

The CARES computer code is also used to compute the model response given the measured free field motion at gage a25. The Beredugo - Novak soil structure interaction model is used. This is done in the three directions X, Y, and V. The unified soil model properties are used with the soil moduli degraded and damping increased depending on soil strain and the degradation model included with the unified soil model. Spectra of the computed motion at the roof and base are then compared with spectra of the recorded motion. The following conclusions are drawn from these comparisons:

1. The correlations between the spectra developed from the predicted and measured response are poor for the X direction motion and good for the Y direction motion.

2. The predicted roof spectra in the X direction are unconservative for frequencies greater than 3.5 cps. The predicted spectra in the Y direction are unconservative in rather narrow frequency bands. It is unlikely that the Standard Review Plan requirements for broadening the spectra and using a range in soil properties would eliminate the region in which the X spectra are unconservative, but likely would eliminate those regions for the Y direction spectra.

3. The forced vibration tests after backfill indicated that the fundamental vibrational mode of the soil / structure system is a rocking mode at a frequency of 6 cps. The fundamental mode for the February and May earthquakes are 5 cps and 4 cps respectively. This reduction in frequency in

3. The forced vibration tests after backfill indicated that the fundamental vibrational mode of the soil / structure system is a rocking mode at a frequency of 6 cps. The fundamental mode for the February and May earthquakes are 5 cps and 4 cps respectively. This reduction in frequency in larger than would be anticipated based on degradation effects on the soil shear modulus.

4. The correlation between the computed vertical motion spectra and the spectra for the measured motion are good at all frequencies except around 5 cps and 25 cps. The predicted values are conservative at 25 cps and unconservative at 5 cps.

5.0 Lessons Learned

The conclusions discussed above were based on studies conducted at CCNY using the CARES computer code. Other members of the consortium have performed similar studies using a range of tools. Synthesis of the results obtained from these studies is ongoing; papers describing the studies have been published in SMiRT 13 and SMiRT 14 by individual consortium members. An attempt is made here to summarize these results based on presentations made at the several meetings held by the consortium over the past six years.

An extensive soils exploration program was conducted by CRIEPI over the duration of the project. The results of these studies were incorporated into "unified soil models" which were used by all participants in making predictions. These data were reviewed by most of the consortium members and the reviews included comparisons of the measured soil properties with those deduced from the model responses measured during the program (especially the forced vibration tests). It can be concluded that the Hualien site was at least as completely analyzed as any nuclear power plant site. In spite of this, significant uncertainties still exist in the soil properties. Much of the measured response data indicates that the "unified soil models" may represent the soil as too stiff. Potential sources of these uncertainties have been attributed to nonlinear effects such as soil separation, and the difficulty in both field and laboratory testing of gravelly soils such as found at Hualien. Many of the measured responses also point to the apparent anisotropic characteristics of the soil properties. This was not considered in formulation of the exploration program and as a result the program sheds little light on the magnitude or source of the anisotropic effects.

Free field calculations were generally done based on the assumption of vertically propagating, horizontally polarized shear waves. The results obtained from other studies were consistent and similar to those reported above. When the horizontal components are transposed to the principal directions excellent agreement between measured data and predictions is found down to the deepest gage (52 m). When the correlation studies are conducted in the experimental (L, T) coordinates very poor correlation is found in the T direction. This lack of correlation is attributed to anisotropic effects.

Two types of computer codes were used to model the forced vibration tests and the response of the model to the seismic events. The first type of code (such as CARES used in the CCNY studies) does not model the soil explicitly but uses lumped parameters (impedance functions that are frequency dependent and are equivalent to springs and dampers) to model the soil structure interaction effects and to connect the structural model (usually a finite element model) to the free field. The impedance functions are usually obtained from libraries of existing solutions for a rigid foundation vibrating in a viscoelastic soil media. This of course restricts the modeling of the soil media to geometries for which solutions exist. Typically, solutions are available for simple soil media to geometries for which solutions exist. Typically, solutions are available for simple horizontal layers in the soil. Variations in the soil such as occur between the backfill and in situ soil must be treated approximately. The primary advantage of this type of code is that they are rather simple to use and are conducive to conducting parametric studies.

The second type of code (such as SASSI) connects a structural model to some discrete model of the soil (finite element or some form of boundary element). These codes (termed continuum codes for reference) have the advantage of treating complicated soil geometries and can include foundation flexibility. These codes have been developed for seismic applications and are different from ordinary structural codes because of the need for special treatment at the boundaries. The horizontal extent of the media included in the problem must be restricted and boundaries placed at some reasonable distance from the structure. Seismic wave reflections occur at these boundaries and disturb the solution at the model location. Various forms of "absorbing" boundaries are used to minimize this effect and allow for the use of reasonable sized problems. These codes are rather difficult and expensive to use.

Predictions of the model response to the before backfill shaker tests made with both types of codes were in good agreement with each other and with the measured data. The predictions of the model response for the after backfill tests were different between the two codes however. The lumped parameter code predicted about 6 cps as the fundamental frequency of the model while the continuum codes predicted a frequency of about 7 cps. This would indicate that the differences between the two codes lies in the backfill model. This is likely caused by the restriction of the backfill model that can be used in the lumped parameter code as discussed above, and the fact that the library of available solutions for the SSI model used on embedded foundations is often based on approximate solutions. The measured data indicated a frequency of about 6 cps, closer to the lumped parameter code prediction. It is surprising that the more approximate codes resulted in better predictions than the more exact continuum codes. There was a general feeling that either the site properties were softer than recommended in the CRIEPI models or that local soft spots were located around the foundation. When the model was changed to reflect these considerations the continuum codes gave predictions that were in agreement with the measured data. It should be noted however that the soils program was not able to confirm either of the two soft conditions.

The predictions of the response of the model to the seismic events were also made with both types of codes. Generally only fair agreement was found between the predicted and measured results. Much better agreement was found when the studies were done in the principal directions than when the studies were done in the (L, T) coordinate system. It is concluded that the transfer functions between some control point in the free field and the structure cannot be adequately computed based on isotropic models of a site which is not isotropic. Tools are not available to treat response predictions as compared with the measured data.

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10. SUPPLEMENTARY NOTES

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11. ABSTRACT *(200 words or less)*

A quarter scale model of a containment structure was constructed in Hualien, a seismically active region in Taiwan. Forced vibration tests were performed both before and after placement of the backfill. The structure and surrounding free field were then instrumented with accelerometers, pressure gages, and displacement gages. Recordings were taken when significant seismic events occurred at the site. This report describes analytical studies that were performed to correlate the measured response with predictions. Soil - structure interaction effects were represented with a lumped parameter model.

The work is summarized in Volume 1 of the 4 volume report. The site characteristics are described in Volume 2. This includes a discussion of the field program used to determine the characteristics. The predictions for the forced vibration tests are compared with the measured data in Volume 3. The correlation is found to be quite good except that the measured data indicated that the site is anisotropic. This is not found from the site exploration phase of the work. Comparisons between the predictions and measured data for the earthquake responses are made in Volume 4. The soil responses compare reasonably well but the in structure responses do not compare very well. This is attributed to the anisotropic site characteristics.

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