Hualien experiment: non isotropic site properties

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ABSTRACT

A quarter scale model reactor containment building has been constructed at a seismically active site in Taiwan (Hualien). The reinforced concrete building model is cylindrical in shape and is embedded about one radius in the soil. The underlying soil is relatively uniform having a low strain shear wave velocity of about 350 m/s. Accelerometers have been placed around the facility and in the near field. The structure has been subjected to forced vibration loadings and seismic events at the site. A review of the data indicate that the site properties are non isotropic. This property is discussed in this paper.

INTRODUCTION

An international consortium has conducted two sets of experiments having the objective of collecting data that can be used to validate the methodologies used to predict the response of nuclear power plant facilities to seismic induced loading. The first set of experiments were performed at Lotung in Taiwan, a rather soft site having a shear wave velocity of about 100 m/s. Data for the second set of experiments were collected at a harder site in Hualien (shear velocity about 350 m/s). Both sites are seismically active. A quarter scale model containment structure was built at each site with both the structure and free field instrumented. Shaker experiments were performed both before and after placement of the backfill. Data was then collected during seismic events occurring near the sites.

Results of the Lotung experiment were summarized by Tang [1]. This paper describes studies done to investigate the non isotropic properties of the site. The results of the shaker tests and the response of the model to seismic events are discussed at other papers at this SMiRT [2], [3].

DESCRIPTION OF EXPERIMENT

The "quarter scale" model reactor containment structure is shown on Figure 1. The structure consists of a 300 mm thick cylindrical concrete shell with a 1500 mm thick roof slab, and a 3000 mm thick basemat. The concrete used for the model has an elastic modulus of 288,000 kg/cm². The fixed base fundamental frequency of the structure is 10.7 cps and the mode shape indicates that the structure behaves as a shear beam (the shell modes are not important). The weight of the structure is 1,425,000 kg. Structural damping is taken as 2% for low stress levels.

The soil consists of gravelly sands with a water table location at the bottom of the basemat. The soil properties were developed by CRIEPI [4] and are summarized in Figure 1. These are
“best estimate” values with the actual data showing considerable scatter. The profile of the excavation made for construction is also shown on Figure 1. Two sets of forced vibration tests were conducted, the first before backfill (FVT-1) and the second after backfill (FVT-2). The recommended soil properties are somewhat different for FVT-1 and FVT-2 because of the change in overburden stress when the backfill is placed. The soil properties used for the seismic response predictions are the same as the FVT-2 properties.

Accelerometers are located on the structure at the top of the roof, mid height of the structure, and on the basement. At each elevation three component instruments are placed at four locations around the perimeter and at the center when possible. Free field accelerometers for the FVT tests are located at the surface along two radial lines at ranges of 1.5 m (BS), 6 m (MS), and 9.5 m (TS) from the outside of the structure. Of course the (BS) and (MS) gages were placed at the surface of the cut for the before backfill shaker tests. The free field surface accelerometers for the seismic event are located as shown on Figure 6. The gages are located 0.5, 1, 2.5, 3.5, and 5 diameters from the center of the model.

The results of both the FVT tests and the model response to seismic loading indicate that the soil properties are likely to be non isotropic. This characteristic is discussed in this paper.

FVT TESTS

The FVT-1 measured responses on the roof for NS and EW shaker loading are shown on Figures 2 and 3 respectively. As may be seen from Figure 2 the NS shaker test resulted in a peak NS roof response equal to 213 µm/t and a peak EW response equal to 130 µm/t. The EW shaker loading resulted in a peak EW roof response equal to 175 µm/t and a peak NS response equal to 53 µm/t. It can also be seen that the peak NS response occurs at a frequency of about 4.1 cps while the peak EW responses occur at a frequency of 4.6 cps. It is also interesting to note that a secondary peak occurs in the NS test at a frequency corresponding to the EW fundamental frequency.

The out of plane response was not expected since all of the model and site geometry appear to be axially symmetric. The most likely cause of this non symmetric behavior would be torsion of the model (which could be caused by some unexpected non symmetry in the model) or some local non uniform soil property beneath the foundation. Torsion can be dismissed by comparing the responses of the different gages mounted on the roof. It can be seen on Figure 2 that gages at locations 8, 14, and 16 have identical responses. Gage 14 is located on the east end of the roof, gage 16 is located on the west end and gage 8 is near the center. Any torsion would result in different responses for these gages. The same result is found for the EW shaker test. Gages 12 and 13 are located on the north and south ends of the roof. Since they indicate identical responses (see Figure 3) there could not have been a torsional response component.

Another potential source of a nonlinear response would be a non uniform soil beneath the foundation. If this were the case one would expect that the horizontal input would cause some vertical response of the model. Figure 4 shows the measured vertical response of the basement caused by a NS roof shaker load. Node 2 is at the center of, node 21 at the north end of, and node 24 is 2/3 the radius out to the south end of the basement. As may be seen the vertical displacement at the center is about 0 while the displacements at the north and south ends are 180° out of phase. This indicates that the vertical deformations are due to rocking of the model about the EW diameter. Node 30 is at the west end of and node 26 is at the east end of the basement. Responses at these two node are also out of phase indicating a rocking response about the EW axis (the out of plane response).

Further evidence of the non symmetric response of the site can be deduced from the vertical shaker test. A symmetric response would require that all of the free field motion is radial. The NS free field responses caused by the vertical shaker load is shown on Figure 5. As may be seen gages located to the east of the model (MS E and BS E) have significant NS tangential response. Note also that the MS N gage (located north of the model) the radial response (NS) is double the non symmetric EW response at gage MS E.

Principal directions are found from the basic data with the soft direction occurring about
340° west of the N direction of the tests. The test and principal directions are shown on Figure 6. Principal directions obtained from the seismic data are shown on the same figure and are discussed below.

The FVT-2 have the same non symmetric characteristics but the effects are much smaller.

SEISMIC RESPONSE DATA

Non isotropic characteristics of the site effect the propagation of the seismic disturbances across the site. This is investigated in this section of the paper. The February 23, 1995 earthquake is used for this analysis, but similar results were found for the January 20, 1994, May 1, 1995, and May 2, 1995 earthquakes.

The earthquake data was collected in the (L,T) coordinate system as shown on Figure 1. Accelerograms of the recorded motions in each direction are transformed into Fourier components. Real and imaginary components are found for each frequency. The real and imaginary components of the frequency dependent transfer functions are then determined from the measured data relating the roof response of the model with the surface input. The transfer functions can be determined from:

\[
R_L = H_{LL} F_L k + H_{LT} F_T k \\
R_T = H_{LT} F_L k + H_{TT} F_T k
\]  

(1)

where, \(H_{LL}\) and \(H_{TT}\) are the transfer functions in the longitudinal and tangential directions respectively and \(H_{LT}\) is the coupling transfer function (L response due to T input). The parameter (k) indicates a particular free field gage. Of course, \(H_{LL} = H_{TT}\) and \(H_{LT} = 0\) for an isotropic material.

The transfer functions (\(H_{LL}, H_{LT}, H_{TT}\)) are then evaluated from a least squared error fit to Eqs. (1) using all of the surface gages outside of the backfill (a13 through a15, a23 through a25, and a33 through a35) and the model responses. It is then assumed that principal directions exist such that the structural response can be written in the principal coordinate system as:

\[
R_X = H_{XX} F_X k \\
R_Y = H_{YY} F_Y k
\]  

(2)

Equations (2) are written using the amplitudes of the transfer functions. The transfer functions in the principal directions (\(H_{XX}\) and \(H_{YY}\)) are then obtained from Mohr's circle. This transformation also gives the angle (\(\theta\)) between the principal (x,y) and the test directions (L,T).

A plot of the angle to the principal axes is shown on Figure 7. It may be seen that the angle is rather insensitive to the frequency and that the average value is about 290°. It is interesting to consider the orientation of the principal directions relative to the site geometry as shown on Figure 6. The principal directions are shown relative to the (L, T) system with the 290° angle taken clockwise from the (L or north direction). It is interesting to recall that the maximum response (softer) principal direction for the FVT-1 test was found to be about 340° counterclockwise from the shaker north (also shown on Figure 6). This is close to the principal directions as found here. Figure 6 may also give some indication of the reason for the non isotropic conditions at the site. Mechanical vibrations radiate out from the plant shown on the figure. These vibrations could have consolidated the soil in the radial direction through the years. Radial lines from the plant are close to the stiffer principal direction at the site.

SUMMARY

The results of the shaker experiments indicated a non symmetric component of the response. This was not expected since the geometry of the model was symmetric. The data also indicated that this non symmetric response was not due to the model since a horizontal loading of the model did not introduce any torsion nor vertical response. It was more likely the result of some nonuniformity in the soil conditions. The non symmetric characteristics of the response was

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also evident at gages located 6 m from the outside wall of the model.

The seismic data was reviewed and also found to have characteristics that would be expected for a site with non isotropic soil properties. Principal directions were determined from both the shaker and seismic response data and found were found to be in agreement.

REFERENCES


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The conclusions presented in this paper represent those of the authors and should not be interpreted as representing the official policy of the U.S. Nuclear Regulatory Commission.
Fig. 2 Roof Horizontal Response Due to NS Roof Shaker Loading (FVT-1)

Fig. 3 Roof Horizontal Response Due to EW Roof Shaker Loading (FVT-1)
Fig. 4 First Floor Vertical Response Due to NS Roof Shaker Loading (FVT-1)

Fig. 5 NS Soil Response Due to UD First Floor Shaker Loading (FVT-1)
Fig. 7 Principal Angle Derived From Roof Data - February 23, 1995