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COMPARISON OF TEST AND EARTHQUAKE RESPONSE MODELING OF A NUCLEAR POWER PLANT CONTAINMENT BUILDING

M. G. Srinivasan, C. A. Kot, B. J. Hsieh

Components Technology Division, Argonne National Laboratory
Argonne, IL 60439 U.S.A.

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Abstract

The reactor building of a BWR plant was subjected to dynamic testing, a minor earthquake, and a strong earthquake at different times. Analytical models simulating each of these events were devised by previous investigators. A comparison of the characteristics of these models is made in this paper. The different modeling assumptions involved in the different simulation analyses restrict the validity of the models for general use and also narrow the comparison down to only a few modes. The dynamic tests successfully identified the first mode of the soil-structure system.

1. Introduction

During the course of an investigation sponsored by the U.S. Nuclear Regulatory Commission on the utility of dynamic testing in the safety assessment of as-built nuclear power plant structures, it was found that the reactor building of Unit 1 of the Fukushima Nuclear Power Station complex in Japan was one of the very few nuclear power plant structures for which response was recorded during both dynamic tests and natural earthquakes. With the objective of evaluating the effectiveness of the dynamic tests for obtaining the dynamic characteristics of the system and/or to form the basis for an analytical model capable of predicting earthquake response, we made a comparison of the analytical models that were devised by previous authors to simulate the different events. As the recorded data were not available to us, this comparison had to be limited mostly to published literature.

2. Description of the Structure

The reactor building of Unit 1, which was subjected both to forced vibration tests and natural earthquakes is about 58 m in height from the base of the foundation mat, the depth of embedment of which is about 14 m below ground level. The building consists of a reinforced concrete structure up to a height of about 43 m from the base, and a steel structure and truss roof above this level. The reinforced concrete structure consists of six floors, including the basement floor. The plan dimensions of the building are 42 m x 42 m at lower sections, and 42 m x 31 m at upper sections. The building is structurally isolated from the adjacent turbine building and radwaste building. The reactor pressure vessel is at the center of the building, surrounded successively by a concrete gamma shield wall, a bulb-shaped steel containment vessel, and a reinforced concrete shield wall. The reactor pressure vessel is connected to the gamma shield wall with horizontal supports and the steel containment vessel is connected to the gamma shield wall by a stabilizer. The containment vessel is also connected to the other reinforced concrete shield wall with shear lugs.

The buildings were designed by both static and dynamic methods of aseismic design procedure. In the static method, a seismic coefficient of 0.48 was used. The dynamic design was based on a seismic analysis of response to a peak horizontal ground acceleration of about 0.18 g [3].

3A Forced Vibration Tests of 1969

A set of forced vibration tests were performed on the reactor building of Unit 1 in November 1969, immediately after the completion of the building [3,4]. A sinusoidal shaker was installed on the 5th floor, the uppermost floor of the reinforced concrete structure. Steady-state excitation in two mutually-perpendicular directions (parallel to the outer walls of the square building) was provided by the shaker. The horizontal displacements at the basement, 2nd floor, 5th floor, roof, reactor pressure vessel, gamma shield wall and the containment vessel were measured. The rotational motion of the foundation mat was also measured. The phase lags of all the responses were measured too. The peak displacement amplitudes in the tests were of the order of 30-40 x 10^{-6} m.

The response data was acquired and analyzed with the "MIK System" [5] in real time, to give frequency response curves. The periods of vibrations were obtained from the peaks of the resonance curves, the first three periods being 0.25 s, 0.17 s, and 0.089 s [3,4]. The mode shapes were obtained by plotting the modal amplitudes measured in the tests.

It appears that no attempt was made to estimate modal damping directly from the test data using methods such as half-power bandwidth or other similar ones. Instead a simulation analysis of the tests was performed employing an analytical model with certain assumed physical constants [3,4]. Apparently the physical constants were adjusted until the analytical resonance curves and mode shapes agreed with corresponding test data. This analysis gave the modal damping, as a percentage of critical damping, to be 33.7, 8.4, and 5.4 for the three modes.

The analytical model employed in the simulation analysis was the same as that used in the dynamic analysis to simulate the response to a 1970 earthquake, and is described in [2]. This lumped-mass, beam-element model consisted of vertical beams for walls and horizontal springs for slabs. The masses were lumped at each floor level. The foundation was divided into three elements with each connected to rotational and translational springs representing the soil. This model also predicted natural periods and mode shapes that agreed well with the ones obtained from the vibration tests.

Mode shapes for the first three modes were also computed by the authors of [3,4] from the analytical model and were compared with the corresponding mode shapes directly obtained from the test data. The comparison is generally very good, and the only significant difference occurs in the first-mode mode shape vector at one location in the roof.

4. Earthquake of 1970

In May 1970, only a few months after the forced vibration tests, a minor earthquake, whose center was located 50 km offshore at a depth of 50 km, occurred. As seismographs had been installed in the building two months before the earthquake, the North-South response of the building at the following six locations was recorded: basement floor, 3rd floor, 5th floor, roof, and the top and bottom of the gamma shield wall. The peak accelerations recorded at any of these locations are not given in [1,2]. However, the response spectra with 5% damping show peaks of the order of 0.03 - 0.06 g at the building locations.

The mathematical model, noted in the previous section, was used in the dynamic analysis to simulate the earthquake response. The measured basement motion was applied as the base excitation to the model in the dynamic analysis. The physical constants of the model (Young's modulus, Poisson's ratio, and a viscous damping coefficient) were adjusted until the computed response agreed closely with the recorded accelerations at different locations in the superstructure.

By considering the damped free vibrations of the analytical model, the authors of [1,2], also determined the natural periods, damping ratios, and the mode shapes for the eight lowest modes. The damped natural period of the first three modes and the damping ratio for the first two modes were identical with the corresponding values obtained for the vibration-test model. The damping ratio for the third mode (the dominant mode of the reactor pressure vessel) was, however, obtained to be 1.6%, in the case of the earthquake response analysis whereas it was determined to be 5.4% in the case of vibration test response analysis. This seems to be a consequence of the unexplained reduction in the value used for the physical constant representing damping in the steel reactor vessel in the earthquake simulation analysis.

The authors show the first eight analytically determined mode shapes in [2]. However, as there was no direct mode shape estimation from the recorded earthquake response, it is not possible to verify the analytical mode shapes by direct comparison.

5. Miyagiken-Oki Earthquake of 1978

On June 12, 1978, about nine years after the vibration testing, the Fukushima plants experienced a strong earthquake. This earthquake, of Richter magnitude 7.4, had its epicenter at about 95 km from the coast of Miyagi prefecture of Japan. The approximate epicentral distance to Fukushima was 140 km. The peak horizontal ground acceleration recorded at the Fukushima plant site was 0.13 g and the duration of the strong gound motion was about 30 sec.

Seismographs and moving-coil accelerometers installed in and near the containment building of Unit 1 of the Fukushima nuclear plant complex recorded the earthquake motions of the soil and the building. The acceleration records used in the simulation analysis to be discussed below [6] were the horizontal N-S motions (the direction parallel to the coast line) obtained from seismographs installed at the following five locations: basement floor, 3rd floor, 5th floor, top floor, and in the soil at an elevation of −40 m (i.e., 36 m beneath the foundation). The maximum acceleration recorded at the basement was about 0.08 g, and that at the fifth floor was about 0.15 g.

Adopting an approach similar to those reported in [1-4], Tanaka and Nakahara [6] performed a simulation analysis of the structural response to the earthquake, instead of applying an inverse method to identify a dynamic model from recorded earthquake response. The mathematical model devised by Tanaka and Nakahara [6] was quite different from that used in [1-4]. In this model, the soil surrounding the building was modeled with much greater detail and refinement. The soil region of the model extended to the level of −40 m, and had a length and width of about 190 m and 63 m, respectively. (The length direction was parallel to the N-S direction). The soil was divided into vertical columns, each of which were idealized as lumped-mass shear beams. The shear-wave speed in the soil was taken to vary with depth. The vertical soil columns were assumed to be fixed at their bottom boundary at −40 m and were interconnected by horizontal springs. The building was represented by two vertical cantilevers, with lumped masses and a horizontal connecting spring at each floor level. Rotational springs also connected the foundation to the soil.

The first part of the analysis was the determination of natural frequencies, modal damping ratios, mode shapes and participation factors. The authors give the periods, participation factors, and modal damping values for ten modes. They also give the mode shapes for the first, third, and fifth modes.

The second part of the analysis was the simulation of response to the 1978 earthquake. The recorded acceleration at the −40 m elevation was used as the excitation for the model. The computed acceleration histories at the basement floor, third floor, and fifth floor were compared with the corresponding earthquake records. According to the authors the agreement between computed history and recorded history is good for the basement floor and the third floor. From the acceleration history figures given in [6], it appears that the fifth floor response is not as well simulated by the model. However, a comparison based on the acceleration response spectra (for 5% damping), shown in [6], seems to indicate that the simulation is satisfactory for response at the basement, third floor, and fifth floor. On the other hand, this comparison indicates that the response of the roof truss at frequencies greater than 4 Hz is not well simulated by the model.

6. Comparisons

As the same mathematical model was not used in all the three investigations, a direct comparison of the physical parameters (e.g., stiffnesses) assumed in the models is not possible. Even the physical constants (e.g., elastic modulus of concrete or the soil) were different in different models because these were in a sense equivalent material properties that were influenced by the modeling assumptions. Moreover the physical constants were adjusted, within the limits of available direct test data [7], to obtain good simulations of dynamic tests or earthquake response. Therefore it is more appropriate to use the modal parameters (e.g., natural periods) as the basis for comparisons.

Table I shows all the natural frequencies identified from the 1969 test data and computed from the two different models used for simulating the 1970 and 1978 earthquakes. The natural frequencies are arranged in deacending order and each mode is identified by a serial number for the sake of convenience. When a mode is identified with a number in the following, it is this serial number of Table I that is meant and not the numerical order of the mode assigned by the authors of [1-6]. To make the comparisons shown in Table I, it was first necessary to compare the mode shapes given by the different models and identify those that are similar as to pertain to the same mode. Since the two simulation models for the 1969 tests and the 1970 earthquake were almost identical, the mode shapes given by both agreed very well. However, only two modes are seen to have been common to the tests and the two earthquakes, and one of these modes (fifth) is considered to be a local mode of the steel structure by all the authors. The other mode (third) appears to be the fundamental mode of the soil-structure system. There is reason to suspect that the first two modes of periods 0.396 s and 0.322 s are not actual system modes.

In the 1978 model a fixed boundary was assumed at the level of -40 m. Tanaka [7] noted that since no such boundary exists, the first mode does not exist in reality. The same reasoning seems to apply to the second mode also. Since in the 1970 model the soil was replaced with soil-springs, no site modes were indicated to exist by this model. Thus it is perhaps correct to assume that the fundamental mode of the system was correctly identified by the dynamic tests. The slight difference in the calculated values of the natural period of this mode between the 1970 and 1978 models is not significant. The fourth mode was not identified in the dynamic tests or shown to exist by the 1970 model. It is possible that like the first two modes of the 1978 model, this mode is also a consequence of the modeling assumptions and not physically realizable. The fifth mode is identified both by the tests and from the the two different models. Here again the slight differences in the natural periods obtained for this mode are not significant. For the sixth and higher modes, the comparison fails. Available information is not sufficient to determine why modes 6-10 (if they really exist) were missed by the dynamic tests.

The comparison of modal damping values, - all of them based on the dynamic characteristics of the models that best simulate each event - is shown in Table II. It must first be emphasized that none of the values in this table were identified directly from the test or earthquake response data through a parameter identification procedure. Thus these values have to be considered to be dependent on the analytical modeling assumptions involved in the three models. It is also for this reason that the comparison is restricted to the modes that were identified or indicated by more than one model.

Considering the fundamental mode of the soil-building system, the large difference between the values of the 1969/1970 models on the one hand and the 1978 model on the other, is explained by Tanaka [7] as due to the difference in the nature of the two models. His explanation may be interpreted to be that the damping value of 33.7% given by the 1969/1970 models ludes all the dissipative effects of soil-structure interaction, whereas the 8.84% given by the 1978 model does not include the energy dissipation within the soil. When one considers that the 1978 model gave modal damping values of 10.08% and 10.15% for its first two modes that are physically unrealizable, the combined dissipative effect of the first three modes of this model might be equivalent to a high damping value indicated by the 1969/1970 models for the first mode indicated by them.

No explanation is available for the differences between the damping values for the other two modes shown in Table II. Though the differences in these modes are not as great, they are not insignificant. Clearly they are a consequence of different physical constants of modeling assumptions made in the different models. From a practical point of view all the above damping values should be treated with caution and only an analysis of the response data might help to identify acceptable damping values.

7. Conclusions

The low-level dynamic tests appear to have successfully identified the first mode and the fundamental period of the Fukushima Unit 1 reactor building. This period has not shifted during the relatively small 1970 earthquake. There is a slight increase in its value, from 0.25 s to 0.26 s for the larger 1978 earthquake. On the basis of available information, it is not possible to determine whether this small increase in the period is due to a softening of the system or due to modeling assumptions. The difference in damping values obtained from the models simulating the different events are significant enough to cast doubt on the practical validity of any of the damping values.

This exercise at comparison of test and earthquake dynamic characteristics has also raised a question as to the utility of simulation models for predicting any responses to excitations other than the ones they were originally devised for. The simulation model for the 1978 earthquake has shown that it could simulate the 1978 earthquake response correctly despite the physically unrealistic assumption of a fixed boundary in the soil at the level of −40 m. It is not known whether this model could successfully simulate the 1970 earthquake or the 1969 tests. Similar criticism also applies to the simulation models of the 1969 tests and 1970 earthquakes in which the choice of the material constants seem to have been adjusted to obtain good simulations apparently without any effort to relate these constants to physical constants obtained from direct tests.

Perhaps more unambiguous and more useful information could be obtained if the recorded response from the tests and the two earthquakes are analyzed using the principles of system identification techniques and applying the same methodology (i.e., based on same modeling assumptions) to all of them.

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Table I. Comparison of Natural Periods

Serial No.	Mode Description	Natural Period (in sec.) from measured frequency response of 1969 Tests (1)	from analytical model simulating response to 1970 Earthquake (2)	1978 Earthquake (3)
1	Fundamental mode of site[3]	not identified	not indicated	0.396
2	not described	not identified	not indicated	0.322
3	Fundamental mode of soil-building system[1,2,3]	0.25	0.25	0.26
4	not described	not identified	not indicated	0.21
5	Local bending mode of steel truss[1,2,3]	0.17	0.18	0.182
6				0.163
7				0.159
8	not described	not identified	not indicated	0.153
9				0.137
10				0.109
11	Local mode of reactor vessel[1,2]	0.089	0.089	
12	Translation & Rocking of foundation mat (second mode of system)[2]		0.077	
13	Local torsional mode of steel truss[2]		0.051	not computed
14	Third mode of system[2]	not identified	0.050	
15	Local translation mode of steel truss[2]		0.048	
16	Local mode of shield wall[2]		0.045	

Table II. Comparison of Modal Damping Ratios

Mode Description Per (1) & (2)	Natural Period (sec.) of Mode	Damping (% of Critical Damping) Simulation Model for 1969 Tests (1)	1970 Earthquake (2)	1978 Earthquake (3)
Fundamental Mode of Soil-Building System	0.25[1,2], 0.26[3]	33.7	33.7	8.84
Local Bending Mode of Steel Truss	0.18[1,2], 0.182[3]	8.4	8.4	6.98
Local Mode of Reactor Vessel	0.089[1,2]	5.4	1.6	not computed