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Comparison of Dynamic Characteristics of Fukushima Nuclear Power Plant Containment Building Determined From Tests and Earthquakes

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M. G. Srinivasan, C. A. Kot,
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Components Technology Division

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ABSTRACT

COMPARISON OF DYNAMIC CHARACTERISTICS OF FUKUSHIMA NUCLEAR POWER PLANT CONTAINMENT BUILDING DETERMINED FROM TESTS AND EARTHQUAKES

by

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

Modal parameters determined from response measured in dynamic tests and from analytical models for simulating the tests and two subsequent earthquakes experienced by the containment building of Unit 1 of the Fukushima Power Station complex in Japan are compared for the purpose of evaluating the effectiveness of the dynamic tests in earthquake response prediction. The tests are found to have led to the correct identification of a fundamental frequency. The lack of agreement between test- and earthquake-determined modeshapes and damping is attributable more to the shortcomings of the simulation models than to differences in actual behavior.

NRC FIN No.	Title
A2217	Evaluation of Testing Methods for Safety Assessment of Nuclear Power Plant Structures

PREFACE

This report presents the results of an investigation conducted for the U.S. Nuclear Regulatory Commission (NRC), Office of Nuclear Regulatory Research (RES), Division of Engineering Technology (DET). The work was performed under a Standard Order for DOE work (FIN No. A2217). The project monitor was Dr. J. F. Costello, NRC/RES; his helpful suggestions and reviews are gratefully acknowledged. The authors also wish to thank Dr. H. Tanaka of the Tokyo Electric Power Co., Inc. for his help in clarifying many issues relating to this comparative study.

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EXECUTIVE SUMMARY

The objective of the study reported is the evaluation of the effectiveness of dynamic tests for characterizing the dynamic properties of a containment building that determine its earthquake response. The containment building in this case is that of the Unit 1 of Fukushima Nuclear Power Station Complex in Japan. It was subjected to forced vibration tests in 1969 and to two earthquakes subsequently: a small earthquake in 1970 and the larger Miyagiken-Oki earthquake in 1978. The containment response to all the three events was recorded and used in studies to characterize the system dynamics.

Natural periods and modeshapes were estimated directly from the dynamic tests. Modal parameters were not directly obtained in the same manner (i.e., by a parameter estimation method) from earthquake response. Instead analytical models were devised by different investigators to simulate earthquake response. A lumped parameter model was used to simulate both the tests and the 1970 earthquake. The physical parameters of this model were adjusted until computed response agreed with the measured response. The modal parameters of the model were also determined.

Two independent analyses were performed to simulate the 1978 earthquake. Though both the analyses used almost the same physical properties of materials, the models employed were very different. In one case the building was idealized as two vertical beams while the site (i.e., soil) extending in all directions up to a relatively large distance was modeled in great detail. In the other case, the superstructure was modeled with a greater number of vertical and horizontal beams while the soil was represented by springs and dampers. Measured ground motion was used as input excitation, either directly or through a convolution analysis. Modal properties for the two models were also determined.

It was difficult to compare even the modal parameters as it was not possible to reconcile the various modes computed for the different models. Only two modes could be determined to be common among the different analyses. It is seen that the dynamic tests correctly identified the fundamental period. Damping determination both from the tests and the earthquakes is so dependent on the modeling assumptions that no conclusion can be drawn about it. As the simulation approach introduces significant uncertainties, this approach should be replaced by direct parameter estimation techniques for obtaining reliable estimates of modal parameters from measured response.

1. INTRODUCTION

The study described in this report was part of a program sponsored by the U.S. Nuclear Regulatory Commission (NRC) to evaluate the utility and limitations of dynamic testing of nuclear power plant buildings for safety assessment. The basis of the evaluation was the actual testing experience relating to large, as-built civil engineering structures, including nuclear plant buildings. In the coarse of a survey of reported testing experience, the results of which were published elsewhere [1,2], 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. If the recorded data were available to the present authors, this would have presented a very good opportunity to determine how reliable the test results would have been for predicting earthquake response. As recorded data was not available to us, the objective of this study was limitedd to an evaluation of the effectiveness of dynamic tests for obtaining the dynamic characteristics of the system that influence earthquake response. The dynamic characteristics are the modal parameters, and the source of information on these was published literature on the test and earthquake response of the building.

2. DESCRIPTION OF THE STRUCTURE

Tokyo Electric Power Company's Fukushima Nuclear Power Station Complex is located on the Pacific Coast in Northern Japan's Fukushima prefecture. The complex has six Boiling-Water-Reactor power plant units. The units were completed at different times during the seventies. The 460-MWe Unit 1 was completed in 1970 and commenced operation in 1971. Units 1 through 5 have a Mark 1 (light-bulb/torus) type of containment and Unit 6 has a Mark 2 type of containment. The plant is founded on a competent soft-mudstone formation with a thickness of more than 300 m. The grade level of the site, at 10 m above sea level, was prepared by excavating a hill of original height of about 35 m above sea level.

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 (Fig. 1). 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.

According to some of the earlier references, [3,4], the soft- or sandy-mudstone soil has a plane wave speed of 1700 m/s, shear wave speed of 610 m/s, and a Young's modulus of 18 t/cm^2. It is not clear what was the basis for determining these values. The analytical models used in various investigations, however, had to use values for shear wave speed, and Young's modulus different from the above to correctly simulate dynamic response. If tests for determining the soil properties were used as the basis for selecting the values of the mechanical parameters, these tests have not been identified or referred to by most investigators.

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].

3. FORCED VIBRATION TESTS OF 1969

A set of forced vibration tests was 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 referred to as N-S and E-W directions) was provided by the shaker. The shaker was capable of operating in a frequency range of 0.2 to 20 Hz, and of producing a peak force of 3 tonnes. 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 measurements included the phase lags of all the responses. The peak amplitudes in the tests were of the order of $30\text{-}40 \times 10^{-6}$ m.

The response data was acquired and analyzed with the "MIK System" [5] in real time, to give frequency response curves. The MIK system obtains the response amplitude and phase through a cross correlation of the response signal and the force signal. 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, presumably because the resonance curves were not smooth enough. Instead a simulation analysis of the tests, employing an analytical model with certain assumed physical constants, was performed [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 respectively for the three modes.

The analytical model employed in the simulation of the test with the excitation along the N-S axis was the same as the one used in the dynamic analysis to simulate the response to a 1970 earthquake, and is described in [6]. This lumped-mass, spring-element model essentially consisted of straight and branched stick models of the building walls. Both bending and shearing stiffnesses of the walls were taken into account. The masses were lumped at each floor level. Springs in horizontal planes modeled the relatively flexible floors. The foundation was divided into three elements with rotational and translational springs representing the soil behavior. This model also predicted natural periods and mode shapes that agreed well with the ones obtained from the vibration tests.

Among the physical constants used in the simulation analysis, the Young's modulus for the soil had to be taken as 45 t/cm^2 in order to get good comparison with the test records. This value is significantly different from the value of 18 t/cm^2, presumably determined directly from field tests. Tanaka, whose investigation of the 1978 earthquake is discussed later, is of the opinion [7] that the value of 45 t/cm^2 for the soil Young's modulus is too large. However, he notes that the authors' use of the higher value in a formula for the determination of the soil spring constant based on static loading yields approximately the same soil spring constants when a lower value for soil Young's modulus is substituted in a formula based on dynamic loading. Also Tanaka considers [7] the Young's modulus for concrete, taken by the authors to be 520 t/cm^2 to be too high, but he does not consider it a significant error as he thinks that this value should be considered in association with the method and assumptions used for modeling the building. Thus it is probable that other material property values employed in the simulation analysis also do not represent the true physical constants, but are actually only mathematical equivalents that depend on the characteristics of the analytical model. If indeed the material parameters had been adjusted without regard to their actual values (determined directly by tests or obtained from engineering practice), the objectivity of the model is open to challenge.

Comparing the periods obtained from the peaks of the resonance curves of the test data with those obtained from the analytical model, we note that the first and third mode periods (i.e. 0.25 s and 0.089 s)agree exactly while the second mode period from test data (0.17 s) is slightly different from that of the model (0.18 s). Since the method used for identifying the periods from test data is probably a visual selection, this difference in the second mode frequency is probably of no significance.

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.

4.0 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 [6,8]. 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. 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. Comparing the physical constants needed to correctly simulate the vibration test response with those for the correct simulation of earthquake response, we find that the only difference occurs in the viscous damping coefficient for the reactor pressure vessel and the primary containment vessel. (The damping coefficient is the ratio of proportionality between stiffness and damping matrices of an element.) The coefficient is 0.00032 s for the vibration tests and 0.00016 s for the earthquake. Whereas it is obvious that the lower damping value gave a better comparison with the recorded earthquake, it is not clear what physical reasoning would justify a lower damping coefficient for the steel vessels in the anlaysis for earthquake response. The authors of [3,4,6,8] do not give any explanation for this.

By considering the damped free vibrations of the analytical model, the authors of [6,8], 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 to the corresponding values obtained for the vibration-test model. The damping ratio for the third mode 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 is clearly a consequence of reducing the values for the physical constant representing damping in the steel reactor vessel and containment, as the third mode is the dominant mode of the reactor pressure vessel.

The authors show the first eight analytically determined mode shapes in [6]. In the first mode (period: 0.25 s) the building and the basemat vibrated in the same direction. In the second mode (period: 0.18 s) the concrete and steel parts of the structures were out of phase, with the amplitude of the steel truss being dominant. The third mode (period: 0.89 s) was predominantly a local mode - the first mode for the reactor pressure vessel. In the fourth mode (period: 0.077 s) the translation and rocking of the foundation mat were very pronounced and the roof vibrated out-of-phase with the base. (The authors [6] noted that this mode may be considered the second mode of the system. This would seem to imply that the mode with a period of 0.18 s is a local mode of the steel truss). The fifth mode, (period: 0.051 s), is a local torsional mode of the steel truss. The sixth mode (period: 0.050 s) has two nodes along the height and is noted by the authors of [6] to be the third mode of the system. The seventh mode (period: 0.048 s) is a local translational mode of the truss. The eighth mode (period: 0.045 s) is a local mode, being the first mode of the gamma shield wall. (Note that the sixth and seventh modes referred to above are incorrectly designated as seventh and sixth, respectively, by the authors [6].)

As there was no direct mode shape estimation from the recorded earthquake response, it is not possible to make a comparison of the analytical and "measured" mode shapes. However, it is possible to make a visual comparison of the mode shapes of the analytical model based on earthquake response with those directly determined from tests. As only three lower modes were determined from the test data (see [3,4]) only the first three lower-mode mode shapes computed analytically (see [6]) are useful for this comparison. The comparison shows that the analytical mode shapes from the earthquake simulation analysis were generally close to the test-determined mode shapes.

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 discussed below [9] 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 buried at an elevation of -40 m (i.e., 36 m beneath the foundation) on the vertical axis of the building. 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 [3,4,6,8] Tanaka and Nakahara [9] 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. However, the mathematical model devised by Tanaka and Nakahara [9] was quite different from that used in [3,4,6,8]. In this model, the soil surrounding the building was modeled with much greater detail and refinement. The soil region of the model had a depth of about 50 m (corresponding to the deepest earthquake observation point at the elevation of -40 m), length of about 190 m, and a width of about 63 m. The length direction was parallel to the N-S direction, the direction of earthquake motions considered. The soil was divided into vertical columns, each of which was idealized as a lumped-mass shear beam. The vertical soil columns were assumed to be fixed at their botton boundary and were interconnected at other nodes by horizontal springs. The building was represented by two vertical cantilevers (having the shear and bending stiffness equivalent to the outer and inner walls), with lumped masses and a horizontal connecting spring at each floor level. Rotational springs also connected the foundation to the soil. The building nodes had horizontal translation (N-S) and rotational degrees of freedom while the soil nodes had only the translational degree of freedom.

The shear wave velocity in the soil was taken to be 180 m/s for the top layer of 5 m depth, 270 m/s for the next layer of 5 m depth and 530 m/s for the third layer of 40 m depth. Tanaka [7] notes that of these values, those for the surface layers were assigned on the basis of measurements made at a different location of the same site and that the average measurement for the supporting mudstone layer was actually 610 m/s but was adjusted down to 530 m/s. Though Tanaka justifies the adjustment by pointing out that it lies within the range of measurements, he implies that the reason for the adjustment was that the adjusted value resulted in a better simulation.

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. The periods for the ten modes are 0.396 s, 0.322 s, 0.260 s, 0.210 s, 0182 s, 0.163 s, 0.159 s, 0.153 s, 0.137 s, and 0.109 s. The corresponding damping ratios, as a percentage of critical damping are 10.08, 10.15, 8.84, 8.12, 6.98, 11.32, 10.27, 10.47, 10.27, and 10.40. They also give the mode shapes for the first, third, and the fifth modes. They note that the first mode (period: 0.396 s) is the fundamental mode of the site model, the third mode is the fundamental mode of the building (period: 0.26 s) and the fifth mode (period: 0.182 s) is predominantly the motion of the steel structure of the containment building.

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 base 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 [9], 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 [9], 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.

The Miyagiken-Oki earthquake response of Fukushima Unit 1 was also used in another investigation sponsored by NRC/RES for correlating the results of the standard Soil Structure Interaction (SSI) methods of analysis with measured response. Miller et al. [10] performed a number of different SSI analyses using lumped parameter and finite element methodologies. As all these studies focused on response prediction aspects of different SSI models, the modal charachertistics of the models were not determined in all cases. In fact modal parameters were determined only for two cases of lumped parameter models. In one of these two cases, the model had only free boundaries for the superstructure with the SSI represented by nodal forces. Because of this the modes of this model do not correspond to the modes of the full soil-structure system. In the other case, the lumped-parameter model of the superstructure was attached to spring and dashpot elements representing the SSI. Therefore the modal parameters of this model can be considered to be that of the full soil-structure system. Thus only this case is pertinent to the purposes of the present study and is discussed below.

The superstructure was modeled as four vertical beams, all lying in the E-W plane (i.e., normal to direction of earthquake excitation) with masses lumped at floor levels and connected by horizontal beams. Horizontal translation in the N-S direction and rotation about the E-W axis were the two degrees of freedom allowed at the nodes. The SSI was accounted for by spring and damper elements connected to the nodes in the base and side walls. Frequency-independent spring and damper constants (termed standard interaction parameters) were determined by averaging the frequency-dependent parameters derived from analytical solutions for steady-state vibrations of rigid foundations resting on a homogeneous elastic half-space.

The material properties used for steel and concrete were identical to those used by Tanaka and Nakahara [9]. However, the shear wave velocity in the soil was assumed to be 490 m/s, which is slightly smaller than the 530 m/s used by Tanaka and Nakahara for the mudstone layer. The damping values for the elements connected to the base was 38.5% of critical for translation and 7.58% critical for rocking; those for the elements connected to the sidewall were 129.2% of critical for translation and 51.8% of critical for rotation.

The authors note that if the structure is taken to be rigid, the natural frequencies associated with SSI occur at 3.6 Hz (period: 0.277 s) and at 10.5 Hz (period: 0.094 s). On the basis of amplifications of the spectral peaks in the measured response, the authors state that the interaction springs are consistent with measured data.

Natural frequencies, mode shapes, and damping ratios are given for the first 12 modes, though actually 15 modes were used in the analysis according to the authors. The natural frequencies of these 12 modes are (in Hz) 2.51, 4.13, 4.44, 8.32, 10.21, 12.47, 12.63, 14.77, 16.46, 17.02, 18.49, and 19.96. The damping ratios (expressed as percentage of critical) for the 12 modes are: 17, 6, 6, 37, 5, 4, 11, 6, 6, 5, 7, and 5. Significant interaction effects were noted to occur at modes 1, 4, 7, 10, and 11.

The response to Miyagiken-Oki earthquake was computed by subjecting the base of the lumped parameter model to input motions. Different cases of input motions were applied to the base though only two are of interest in the present context. In one case the free field motion measured directly under the basemat was used as input. In the other case, the input motion used was derived from a convolution analysis where the motion measured at the -40 m elevation was propagated up through the soil to the basemat location. In calculating this input motion, a soil damping of 10% of critical was assumed. The other cases considered by the authors of [10] are not considered here because for these cases (in which the interaction spring and/or damper parameters were varied to obtain a best fit between computed and measured data) no corresponding modal parameters of the lumped-parameter model were given.

Comparisons of computed spectra with measured spectra for the four measurement locations in the building are shown. For the case where the measured pulse under the basemat was used as input, the agreement is good at the basemat but significant differences are noticed at the other locations. Specifically, the frequencies at which spectral peaks occur in the computed and measured spectra are noticeably different. For the other case where the calculated pulse was used as input, the comparisons at all four measurement locations is even less satisfactory. The authors conclude that the standard interaction parameters are correct while the convolution analysis is responsible for the differences. But the discrepancies noted in the comparisons of the former case do not support this conclusion in our view. Though the computed responses may be considered as acceptable approximations of measured responses, the lack of coincidence of the spectral peaks leads us to question whether the modal parameters of the lumped-parameter model with standard interaction parameters are actually the best approximations to the modal parameters of the physical system.

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 some cases 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.

A difficulty arises even in comparing the modal parameters. The different models gave rise to modes that are not the same. This difficulty is compounded by the fact that without analyzing the numerical data for the mode shapes, it is not possible for the present authors to decide whether two modes determined from different models are the same on the basis of a visual examination of the mode shapes sketched. Therefore the original authors' description of the mode, the frequency at which the mode occurs and the sketch of the mode shape were all taken into account in making the comparison. Table I shows the natural periods identified from the test data, and the natural periods and damping ratios computed for the four different analytical models simulating the test and earthquakes. The modes are arranged in the order of descending periods and each mode is identified by a serial number for the sake of convenience. It must be noted that in almost all cases the serial number is different from the mode number assigned to the mode by the original authors. When a mode is identified by a number in the following, it is this serial number that is referred to.

A glance at Table I will demonstrate the difficulty noted above in making this comparison. The number of modes given by various authors ranged from a minimum of 3 to a maximum of 12. But since the modes obtained from different models could not be identified to be the same except in a few instances, there are actually a total of 27 modes, with periods varying from 0.398 s to 0.045 s, that were determined from different models. As is clear from the table there is a great deal more difference than similarity between the modal parameters of the various models.

Since the two simulation models for the 1969 tests and 1970 earthquake were almost identical, the mode shapes for the two models agreed very well. Only two of the mode shapes (i.e., 3 and 7) obtained by Tanaka and Nakahara [9] can be identified to be similar to two determined by Muto et al.

[3,4,6,8]. Even though the model of the former is very different from that of the latter, the description of three of the ten mode shapes given by the former made it possible to identify these as common modes. Tanaka also confirmed this subsequently [7]. As for the mode shapes given by Miller et al. [10], it was not possible to relate any except their first mode with any of the modes determined by others. Miller et al. [10] stated that their first mode was identical to that determined by Tanaka and Nakahara. Even for their modes with a period very close to that determined by any one of the other authors, the mode shapes do not seem to agree.

Furthermore there is strong evidence to consider the first two modes given by Tanaka and Nakahara [9] to be fictitious (in the sense that these are merely the consequence of modeling assumptions) and not actual modes of the physical system. The soil boundary at the level of -40 m was assumed to be fixed in their model. Tanaka admitted [7] that "since the actual site soil profile does not have the distinct boundary at the corresponding level (i.e., at -40 m), the above mode does not actually exist." While no such admission regarding mode 2 has been made, it appears to us that this mode also is peculiar to the site model and does not exist for the physical system.

It is somewhat surprising that Miller et al. [10] also determined a mode at about the same period (i.e., mode 1) and claimed it to be the same as those given by Tanaka and Nakahara, despite the lack of a fixed soil boundary in the model of Miller et al. [10]. As Miller et al. do not model the soil as a continuum, their mode shapes contain only the superstructure deformations. On the other hand Tanaka and Nakahara [9] have only two beam elements to model the superstructure but an elaborate set of elements to model the site. Thus it is difficult to be sure that mode 1 of [10] is the same as mode 1 of [9].

Thus it would be appropriate to exclude the modal parameters of Miller et al. [10] from further comparisons due to lack of any common ground between them and the rest. Even among the rest, the comparison has to be limited to modes 3 and 7 which were the only ones determined independently by two different models by two different set of investigators. There is agreement that mode 3 is the fundamental mode of the soil-building system and that mode 7 is a local bending mode of the steel truss [3,6,9]. Comparing the natural frequencies for these modes, the slight difference in the calculated values between [6] and [9] is not significant. The change in period of mode 7 from 0.17 s (identified from test data) to 0.18 s (determined from models) could also not be considered as representing any true softening of the system. Thus the tests may be considered as successful in identifying the periods of these two modes.

Concerning the damping ratios, 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. These results probably are directly related more to the assumed damping properties of the materials than to the other aspects influencing energy dissipation. Considering the fundamental mode of the soil-building system, the large difference between the values of the 1969/1970 models [3,6] 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 includes all the dissipative effects of soil-structure interaction, whereas the 8.84% given by the model of [9] does not include the energy dissipation within the soil. When one consideres that the latter 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 common mode. From a practical point of view all the above damping values should be treated with caution and only an analysis of the measured 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. One of the simulation models for the 1978 earthquake [9] 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 -40m. It is not known whether this model could successfully simulate the 1970 earthquake or the 1969 tests. The other analytical model [10] response to simulating the 1978 earthquake was not even as successful in simulating this earthquake. 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 without any effort to relate these constants to physical constants obtained from direct tests.

The objective of this study would have been better achieved had the modal parameters been determined from measured response through a parameter estimation analysis. Employing the parameters of simulation models was resorted to only because the data were not available to us. Based on this experience the use of simulation analyses in lieu of parameter estimation methods is not recommended.

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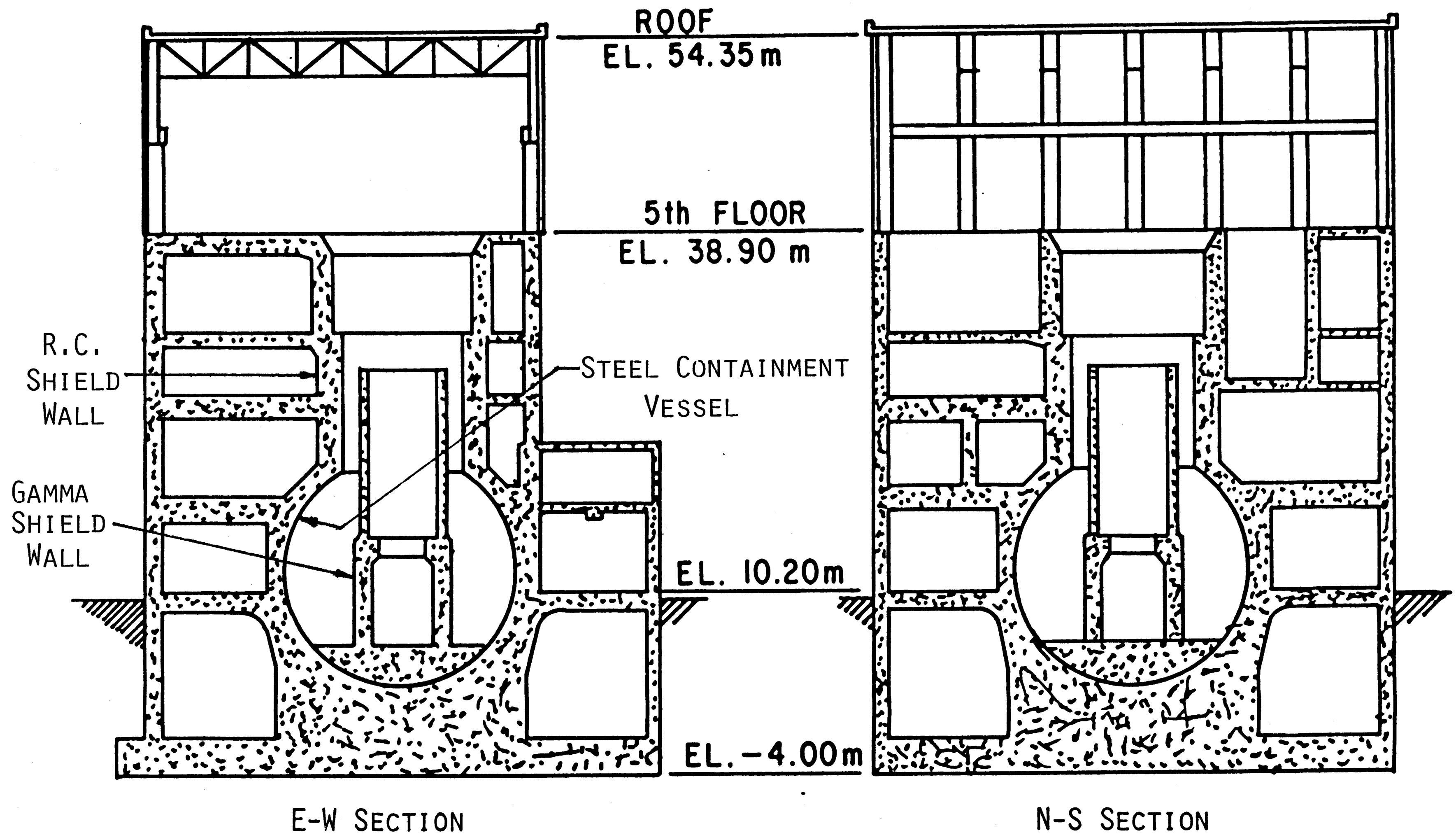

FIG. 1. FUKUSHIMA UNIT 1 CONTAINMENT BUILDING

Table I. Comparison of Modal Parameters

Serial No.	Description/characteristics of mode as given by authors of noted reference	Natural Period, in seconds: from measured frequency response of 1969 tests (Ref [3])	Natural Period: from analytical model simulating response to earthquake of 1970 (Ref [6])	Natural Period: earthquake of 1978 (Ref [9])	Natural Period: earthquake of 1978 (Ref [10]*)	Damping, % of Critical: 1969 Tests (Ref [3])	Damping: from analytical model simulating response to Earthquake of 1970 (Ref [6])	Damping: Earthquake of 1978 (Ref [9])	Damping: Earthquake of 1978 (Ref [10]*)
1	Fundamental mode of site [9], significant interaction effects [10]**	–	–	0.396	0.398	–	–	10.08	17
2	–	–	–	0.322	–	–	–	10.15	–
3	Fundamental mode of soil-building system [3,6,9]	0.25	0.25	0.26	–	33.7	33.7	8.84	–
4	–	–	–	–	0.242	–	–	–	6
5	–	–	–	–	0.225	–	–	–	6
6	–	–	–	0.21	–	–	–	8.12	–
7	Local bending of steel truss [3,6,9]	0.17	0.18	0.182	–	8.4	8.4	6.98	–
8	–	–	–	0.163	–	–	–	11.32	–
9	–	–	–	0.159	–	–	–	10.27	–
10	–	–	–	0.153	–	–	–	10.47	–
11	–	–	–	0.137	–	–	–	10.27	–
12	Significant interaction effects [10]	–	–	–	0.120	–	–	–	37
13	–	–	–	0.109	–	–	–	10.40	–
14	–	–	–	–	0.098	–	–	–	5
15	Local mode of reactor vessel [3,6]	0.089	0.089	–	–	5.4	1.6	–	–
16	–	–	–	–	0.080	–	–	–	4
17	Significant interaction effects [10]	–	–	–	0.079	–	–	–	11
18	Translation and rocking of basemat (second global mode of system) [6]	–	0.077	–	–	–	70.4	–	–
19	–	–	–	–	0.068	–	–	–	6
20	–	–	–	–	0.061	–	–	–	6
21	Significant interaction effects [10]	–	–	–	0.059	–	–	–	5
22	Significant interaction effects [10]	–	–	–	0.054	–	–	–	7
23	Local torsional mode of steel truss [6]	–	0.051	–	–	–	2.4	–	–
24	Third global mode of system [6]	–	0.050	–	–	–	35.9	–	–
25	–	–	–	–	0.050	–	–	–	5
26	Local translational mode of steel truss [1]	–	0.048	–	–	–	2.9	–	–
27	Local mode of shield wall [6]	–	0.045	–	–	–	4.3	–	–

*The analytical model is the lumped parameter model with standard interaction parameters
**Authors of [10] state this mode is identical to that given by [9]

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Modal parameters determined from response measured in dynamic tests and from analytical models for simulating the tests and two subsequent earthquakes experienced by the containment building of Unit 1 of the Fukushima Power Station complex in Japan are compared for the purpose of evaluating the effectiveness of the dynamic tests in earthquake response prediction. The tests are found to have led to the correct identification of a fundamental frequency. The lack of agreement between test- and earthquake-determined modeshapes and damping is attributable more to the shortcomings of the simulation models than to differences in actual behavior.

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