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ON RESPONSE ANALYSIS FOR STRUCTURAL DESIGN AND ITS RELIABILITY

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SUMMARY

This paper deals with the reliability of response analysis to predict the behavior of structures under future strong earthquakes.

Although the accuracy of response analysis of structures seems to be increasing recently, this accuracy means only those for given time-history records of the past strong earthquakes which we concern.

For the structural design, it is necessary to predict their behavior under future strong earthquakes. But it is not possible to predict the time-history record of a particular future earthquake in a sense of deterministic process treatment. We only can predict in a sense of stochastic process approach.

If we treat the response analysis in a stochastic sense, we must make clear the causes which give their effects on the reliability of the analysis. The author discusses on some practical ways of the response analysis for structural design in this point of view through his experience on theoretical analysis, computer simulation, model experiments on shaking tables, and fields experiments to natural earthquakes.

Some portion of the discussion was presented at the Fifth Conference on Earthquake Engineering in Rome already, but the author tries to rearrange his knowledge to suit the practical procedure. And his data after 5WCEE show that the response factors mainly depend on the magnitudes of earthquakes and the location of their test.

He also refers to two new problems. One is the response analysis of structures including piping systems to vertical ground motion. The other is elasto-plastic analysis. The latter analysis which we are doing is time-history dependent usually. He discusses the basic idea abandoning to utilize specific earthquake records, for example El Centro earthquake and so on.

1. Introduction

There has been many discussions which we should use time history analysis or response spectrum analysis for the aseismic design of Class A (or I) structure in Japan for these ten years. It was originated from the discussion between two groups, the PWR design group and BWR design group. The PWR's group more insisted on response spectrum analysis, because main portions of the PWR plant are more rigid than those of the BWR. But the Japanese practice[1] of nowadays has been more weighted in time history analysis. Why is the time history analysis better than the response spectrum analysis? The author thinks that there is no eminent reason to say it. Of course, the most exact solution of the response of a structure to a particular earthquake, for example, El Centro earthquake 1940 can be obtained by the time history analysis. However, has the next strong earthquake in a particular site the same time history as that of El Centro earthquake? The answer is "no". The patterns of the earthquakes observed in the same site fluctuate very much. As the author will mention in the latter chapter, we sometimes find the very similar ones to each other, but we can not expect it usually. So in the viewpoint of the accuracy of the analysis, it seems not to be necessary to use time history analysis.

If we need the time history analysis for systems, for example, a multi-degree-of-freedom system or non-linear system, the time history data should not be limited only one or a few sets like El Centro earthquake and so on. As the author will show it to the reader in the next chapter, the result of response analyses to several sets of pseudo-earthquake data fluctuates very much. At least ten sets of data seem to be necessary for the accurate analysis. Unless otherwise, the modal analysis method with response spectrum should be used. To non-linear analysis, there is no effective method to utilize the response spectrum for complicated systems like piping systems. So, what kind of time history data is the best to the non-linear analysis? One of the extreme cases is Drenick's least favorable excitation data[2] . Although there are many papers to produce pseudo-earthquake data, most of them give the data as some samples from their family. What we want here is the data by which we can complete our response analysis through only one response calculation. The least favorable earthquake may give the upper limit response. The way of generating the time history data to fit the particular response spectrum was suggested by Rizzo[3] and others at the 2SMiRT. Even if we use this practice, still the problem remains. Because the phase relation of each component is still left in designer's hand. In his paper that of El Centro earthquake was used. If we give it statistically, then the result becomes again a sample from the family fitting to the particular response spectrum.

The result obtained from the stress analysis of structures under a earthquake loading is only the guidpost to establish their aseismic design. The error of estimating the magnitude of the Design Basis Earthquake gives the large deviation to the design value of acceleration, then to that of the resulting stress, so there is no meaning to discuss, for examples, 2% excess or 5% less of the stress resultant over the allowable stress.

2. Mechanism of Fluctuation of Responses

The author had been trying to look out the practical scheme of evaluating the response of a piping system bridged between two independent buildings. Through that study, the author obtained the responses to one hundred pseudo-earthquakes from a low-frequency noise generator as shown in Fig. 1. As the author refered in the several reports, the dispersion factor, the ratio of the standard deviation to the mean is very large. And their moving averages are as shown in Fig. 2. The shapes of curves and their dispersion factors are quite independent to their parameters, for the case of Fig. 2 example, the ratio of the eigen period of a supporting building to that of the other. The average response factor fluctuate where the number of earthquakes is less than ten, and over twenty the average seems to be settled. The fluctuation in the case above-mentioned is based on the fact that the pseudo-earthquakes belong to stochastic random process. Besides this mechanism, there are several reasons to cause such fluctuation in response analysis. That is:

1) how do the nest of sources of earthquakes distribute?
2) what path do earthquake waves transmit through?
3) how do the responses fluctuate by the sampling effect from the family of earthquakes belong to the same nest?
4) how are the responses affected with initial conditions of a responding system for each earthquake?

In addition to these there may be another discrepancy which comes from the estimation errors of vibration characteristics of the responding structure itself[4) 5)].

Except the fluctuation caused by sampling effect shown item 3), the other mechanisms are data oriented matter. So the study should be based on the observation of responses to natural earthquakes.

3. Fluctuation Caused by the Non-stationarity of Earthquake Waves

As the author reported in several previous reports,[6) 7)] the effect to the fluctuation of response spectrum induced by the non-stationarity of earthquake waves is large. The histograms in Fig. 1 show the effect. Their non-stationarity can be devided into three characters, at first the length of waves is finite, secondarily, the power of waves is changing by time, and thirdly the instantenious power spectrum is also changing by time. Among those three, the finite length is most strongly related to the fluctuation problem of the response factor. If the length of waves is shorter measured in their dominant period, or if they contain only several dominant wave peaks among their duration, then the response factors may deviate largely from their mean. But that of waves is longer enough to make the responded system stable, the response factor of each earthquake does not fluctuate compare to the mean of responses. The theory is developed based on the statistical characteristics of total wave energies of input and output waves.

In the case of the model shown in Fig. 1, the duration was 20 times of the time constant of the pseudo-earthquakes. Under such a condition the dispersion factor of the resonated single-degree-of-freedom system is 0.21 and those of flexible systems that is from 0.21 to 0.29 according to their eigen period. On the other hand, for rigid systems that is about 0.1. Here the critical damping ratio of the system is 0.07, and corresponds to that of rather rigid concrete structure. For light damped systems the dispersion factor is much bigger than the value above mentioned, for example, that of a system of 0.007 is 0.42 in resonance. Other figures are shown on Table 1.

Of a multi-degree-of-freedom system the freedom of the distribution of the wave energies n_i and n_o decrease along with increase of its degrees-of-freedom. Therefore at that situation the variance of the output waves increases, that is, the wave form approaches to sinusoidal one.

4. Fluctuation of Response Factor to the Earthquakes from the Same Nest

As shown by □ dots in Fig. 5 later, sometimes response factors of various points of the plant took the same values to successive earthquakes. In the case of Fig. 5 there were three successive earthquakes from the same region for an hour. The responses to these two earthquakes were very similar to each other. However, those to other earthquakes from the same region were not equal to them and much scattered.

The same things[7] happened on earthquakes in El Centro City. Two successive earthquakes were occured in the early morning of Feb. 9, 1956. Their auto-correlation functions are quite similar to each other shown in Fig. 3. But that of the famous El Centro Earthquake (top of Fig. 3) is more white, and lacks components of longer periodic waves.

To observe the responses of a model plant, that is, a three story building equipped with a single-pressure-vessel-and-two-piping system and other equipment models to Matsushiro Earthquake Swam, it was built in Nagano City 25 km North from the epicenter region[8] [9] . About sixty records had been recorded over the period from January 1968 to March 1969. Among these sixty records, the author picked up eleven earthquakes which are judged to be originated from the same nest. The maximum values of the acceleration of these earthquakes from 1.09 gal to 25.0 gal. At first, after normalizing them the auto-correlation functions of those earthquakes were calculated separately, and by averaging them the mean auto-correlation function was obtained. The standard deviation has no tendency to diverge with the time. By transforming it into power spectrum by Wener-Khinchin method, the estimated mean power spectrum $E\{A_{SN}(\omega)\}$ in Fig. 4 was obtained.

The dotted line shows the standard deviation of power spectra of eleven earthquakes. At the peak f_{g1} the ratio is about one third, and it is the same order to the relative dispersion factors which the author has been discussed in the previous chapters.

The responses of the model building and equipment were scattered very much. The response factors of the acceleration of the third floor to the basement had been from 1.4 to 6.9. And those of the equipment and pipings were more scattered, for example, out-of-plane motions of a Z-shape piping system responded in the factor from 4.4 to 22.8. As we could foresee the results which would be obtained in Chiba several years later from these results, there is very low possibility that the expected future earthquake will behave in the same way that of the past strong earthquake which we are using for our design analysis.

5. On Pseudo-earthquake Generator

For the analog simulation of which results shown in Fig. 1, the author used the pseudo-earthquakes. The generator of those earthquakes was a filter expressing a dominant frequency of the ground motion and a low-frequency noise oscilator. There are two models for it. One is a constant acceleration type expressed in eq. (1), and the other is a constant velocity type in eq. (2). To generate the pseudo-earthquakes for design, although both types are using, the constant acceleration types are dominant.

The author tried to fit both models to the mean of estimated power spectra of the eleven earthquakes(in Fig. 4). With the constant spectrum or white noise in acceleration the transfer function of the ground filter is as follows:

$$H_g(S) = \frac{2\zeta_g\omega_g S + \omega_g^2}{S^2 + 2\zeta_g\omega_g S + \omega_g^2} \qquad (1)$$

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and their gain characteristics shown in Fig. 4 as a chain line with the mean of the estimated power spectra. Compare to the standard deviation of that shown as a dotted line, the error of this model on the slope is comparable.

On the other hand, the ground filter with the constant spectrum in velocity is expressed as follows:

$$H_g(S) = \frac{-S}{S^2 + 2\zeta_g\omega_g S + \omega_g^2} \qquad (2)$$

The gain characteristics of the later case, shown as a thick solid line in Fig. 4, fit very well the mean of estimated power spectra.

Housner showed that the mean response spectrum is almost flat in the velocity response in his early paper. Also the law of the energy distribution in a multi-degrees-of-freedom system, the author thinks that the velocity constant type model is more natural to generate the pseudo-earthquake. And this discussion will be related to the phase problem in Chapter 10.

6. Response Observation in Chiba Field Station

In Chiba Field Station 50 km away from Tokyo to the east, the observation of responses of a chemical engineering plant model to natural earthquakes has been continuing. The model consists of a monolithic building, a tower type tank, a hanged tank, a pillow type elevated tank and two pipings connecting these two tanks. The vibration characteristics of the main items are as shown on Table 2. Since Sept. 1971 horizontal response and also since Oct. 1973 vertical responses were observed. The distributions of their vertical responses are shown in Fig. 5. The behavior of vertical responses of some items like pipings and a horizontal pillow type vessel is quite similar to that of responses to horizontal ground motion. Their statistical data are summarized on Tables 3 and 4. There are no eminent differences between response to horizontal motions and that to vertical motions. Especially the pipings behave very dynamically to both directions, and the mean of response factors to vertical motions is larger than that to horizontal motions. The relative dispersion factors of flexible systems are around 0.3 $\sim$ 0.4 and that of the horizontal elevated tank is over 0.5. Their tendency and values are agreed with the theory. Response factor is usually said to be governed by the duration of ground motions and their spectrum, and these two parameters are related to their epicenter distances and their magnitudes. To check it, earthquakes were classified to near field ones (A, A', AB and T, of which hypocenter distances were from 50 km to 100 km) and others. Responses of the hanged tank are plotted separately to earthquakes in each categoly. In Fig. 6 the responses to near field earthquakes are shown. Here the abscissa is the surface ground acceleration instead of the magnitude, because the hypocenter distances of these earthquake were almost same range in each categoly. There seems to be a tendency of decreasing of the response factor according to increasing of the ground acceleration. On the other hand, the opposite tendency can be found on the response factors to other groups (in Fig. 7).

7. Abnormally High Amplification

On Tables 3 and 4, the maximum response factors are shown for four items. Those values are extremely large, and most on Table 3 are out of the range of 3σ. Epicenters of six of those showed abnormal amplification belong to A and A', and those of two belong to B out of the fifteen abnormal response cases to horizontal ground motions. The wave forms from the nest A are usually beating strongly, and those from the nest B are the type of narrow banded white noise. In some cases, the amplification factor is nearly equal to that to stationary sinusoidal input. The wave forms of four unidentified cases are also similar to those of type A. But we should pay our attention to the fact that even the cases of type A, most of responses are not so high out of the 3σ range. The abnormally high response factor seems to come from the simple resonance phenomenon of structures to the ground motion but it is rather rare event. In Fig. 8, the very typical example is shown. In this case, the horizontal tank was resonated to the high frequency component of the vertical ground motion. The figure 5.88 on Table 3 is this case.

In conclusion, such an abnormal response depends on the vibration characteristics of the waves which come from the particular nests. If we consider a rare case, we should take such high response factors which correspond to its resonance into our account.

8. Vibration Experiment of the Model Building and Pipings on Shaking Table

A three-story building equipped with a vessel and two pipings which was similar to the model built in Nagano City was rebuilt on a shaking table. A constant force type hanger, and/or oil dampers were mounted on pipings. As shown on the Table 5 the effects of the critical damping ratio to the relative dispersion factor is very clear. Through this experiment, also it became clear that the fluctuation was induced by the non-linearity, especially friction of the system. The effect appeared in its higher mode mainly. The model were shaken by the same earthquakes in ten times and the responses of out-of-plane motion of the piping with a constant force type hanger were observed. The fluctuation of them were significant as shown on Table 5, though that of the building was small. Most of steam pipings in such plants are covered by thermal insulator. If it is solid type, it gives frictional damping to the pipings, and this effect becomes more significant than the author's experiment. Also this effect may be stronger in those to random type ground motions than to narrow banded type ground motions. This is opposite tendency to the abnormally high response problem.

9. Design Spectrum

As the author has been discussing in the previous sections, the use of time history analysis for the design is not practical except the case which he is going to discuss in the following section. Then how shall we decide the design spectrum.

The author tries to become clear the relation of the error of estimating response value to the increase of the risk of fracture caused by exceeding earthquake loadings caused by the error.

The standard deviations of response factor are 30% and 50% of the mean in direct response spectrum and floor response spectrum respectively. Here we assume that the error of estimating the magnitude of design basis earthquake is ±0.3 and it corresponds to the error of range 2σ. And the errors corresponding to 3σ of DRS and FRS give 90% and 150% over response respectively. Then (Mean response + Response Fluctuation and Error caused by magnitude estimation) is 240% and 300%, where the error of the magnitude ±0.3 gives 40% error in ground velocity or acceleration. If the stress caused by earthquake loadings is 20% of the total stress or the allowable stress, then 18% and 30% over stresses may cause at the probability of $(5 \times 10^{-2}) \times (3 \times 10^{-4}) = 1.5 \times 10^{-4}$.

The relation of the increase of the probability of fracture to over-stress. Ikeda summarized as a chart it in his report[10] from Udoguchi and Tagart's papers. From the chart the author assumes that 20% over-stress would give the increase of the probability by approximately 10^3. These over loads caused by error of estimation increase the probability of the fracture as shown on Table 6, and finally the risk of fracture including the error of estimating the magnitude increase by ΔR , that is, 2×10^0 and 2×10^2 when mean direct response spectrum and mean floor response spectrum are used for design respectively. These figures mean that mean direct response curve is good enough as the design criteria, but FRS should have the allowance of 1σ width to their mean in the case that the stress caused by earthquake loadings is 20% of the total load.

However, if it is 50%, ΔR becomes 4.5×10^6 and 1.5×10^{11}. So when we use DRS for the design of such members, we should add at least the width of 3σ to the mean. And to use FRS, we should expect high uncertainty for the design result. The value of 1.5×10^{11} is not so exact because the relation which the author assumed is very rough. Anyway, to improve the accuracy the width more than 4σ should be added, even if we can omit the uncertainty of estimating the magnitude. The author should emphasize that the response analysis has extremely low reliableness, and we should cover the structural reliability by the over-all ductility of the structure and other ways. Some theoretical response curve is also useful. In Fig. 9 the curves obtained by Sato[12)] according to the theory of extreme are shown. These curves are mean floor response spectra, and FRS to El Centro and other earthquakes are also shown. The width given on a curve shows the range of 30% and 50% relative dispersion.

10. Time History Data

For the non-linear analysis, we need input waves sometimes. Under the condition of Emergency Condition or Faulted Condition, the structure behaves in plasto-elastic way. If the structure is expressed in a canti-lever type model like a simple building, there is a way to use a some modified power spectrum. However, for the analysis of piping systems under such conditions we should use time history data.

The way of synthesizing time history data from the given response spectrum[3)] was presented at 2SMiRT by Rizzo and others. This method is practical, but there is a problem in the view-point where the author is discussing. To give the phase relation between each frequency component, they used that of El Centro earthquake. The phase relation is very important to non-linear problems, especially to evaluate the accumlative damage of material by fatigue. Although most of the phase relation of pseudo-earthquakes are given as uniformly distributed random one, through the analysis of the mechanism of the fault movement the assumption that each movement at the base rock would be a step motion seems to be more natural than the assumption of an uniform random distribution. The author reported that the acceleration waves of ground surface consisting five layer model responded to a step movement at the base gave the response curve and other characteristics very similar to those of natural earthquakes[11)] (in Fig. 10). The author feels the necessity of our continuous study in this point.

Drenick presented the idea of the least favorable excitation in 1968 at the US-Japan Joint Seminar on Applied Stochastics[1] . He was working in an area of control engineering, and his idea came from that field. This idea is as follows:

To define a norm, for example, total energy E like

$$E = \int x^2(t)\,dt \qquad (3)$$

then to set an upper limit E^* .

It is desired to find the maximum excursion of $|y(t)|$ which the structure will experience in response to any one of the exitations for which $E < E^*$, regardless of the duration of the earthquake. The solution for well damped structures in this case is inverse of the step response of the system, that is, $h(-t)$ and it begins at $t = -\infty$ and ends at $t = 0$. There need some modification for light damped structures, and it has not developed enough for practical use. The author has never applied this idea to the design analysis, however, this idea is not stochastic, but deterministic. So it seems to have the possibility to use for the non-linear response analysis under controlled design criteria.

11. Conclusion

The author makes the following facts clear in accordance with the author's previous papers;

1) The fluctuation of responses of a structure to earthquakes is very large. Their standard deviation is 30% of the mean of a sindle-degree-of-freedom system and is 50% of the mean of a two-degree-of-freedom system.

2) Causes of such fluctuation mainly come from the stochastic nature of the mechanism of generating and transmitting of earthquake waves. And it is very difficult to reduce their dispersion by categolizing the origin of earthquakes, the magnitude, the duration and so on.

3) The error of estimating the magnitude is said to be ±0.3, and this value corresponds to 40% error of the ground velocity and acceleration.

4) In the extreme case that realization of both magnitude and response factor runs to the extreme, the error might be 140% of the mean. If the stress caused by earthquake loadings is 20% of the total load, and if a member is designed as 100% of the allowable stress, the error mentioned above cause 28% over-stress. This over-stress increases the probability of failure of the member by 1.4×10^4. If we assume that the risk of occuring such an extreme case would be 1.5×10^{-4}, the total risk of fracture by the extreme case of the future strong earthquake would be almost twice.

5) For the design response curve, no width should be added to the mean direct response spectrum, and 3σ of width to the mean floor response curve, if we can assume that the stress caused by earthquakes is 20% of the total load or the allowable stress. If it is 50%, it is very difficult to obtain the reliable design result, so we should pay more attention to keep the system ductility or try to decrease the ratio.

6) Analysis using a single or a few time history data gives only the result which has a possibility to have a big error. So time history analysis is not recommendable for the design analyses except those in non-linear regions, for examples, elasto-plastic, frictional and so on.

7) Although on generating time history data from a given response spectrum, the consideration of phase relation between each component is very important, this technique has not developed well. The idea of the least favorable earthquake has the possibility to develop for this purpose.

12. Acknowledgement

The author owes to much the co-authors of the previous papers which he refered in this paper. He wishes to express his gratidues to them for their valuable co-operations and discussions, and also his many thanks to the members of the sub-committee in the Japan Electric Association for the discussion on the risk assesment.

He also indebted to Mr. Tsutsumi, Mr. Shigeta and Miss Ogino for having reviewed and prepared the manuscript and drawings.

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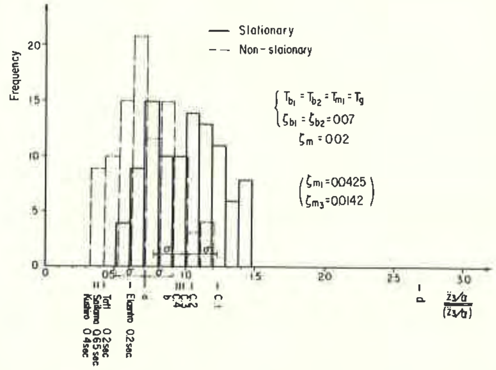

Fig. 1 : Histograms of Response Factors of Bridged Piping to 100 Pseudo-Earthquakes

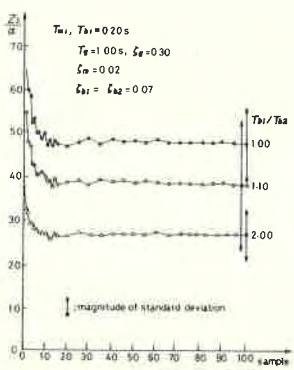

Fig. 2 : Moving Average of Response Factors of Bridged Piping to 1 ∿ 100 Pseudo-Earthquakes

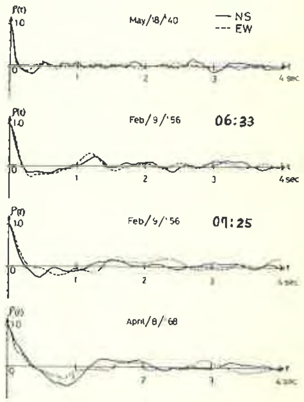

Fig. 3 : Auto-correlation Functions of Four Earthquakes in El Centro City

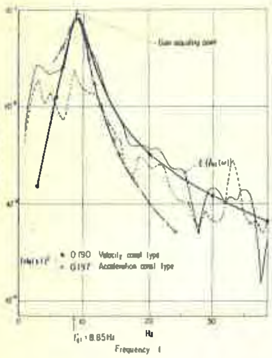

Fig. 4 : Mean Power Spectra and Transfer Function of Estimated Model

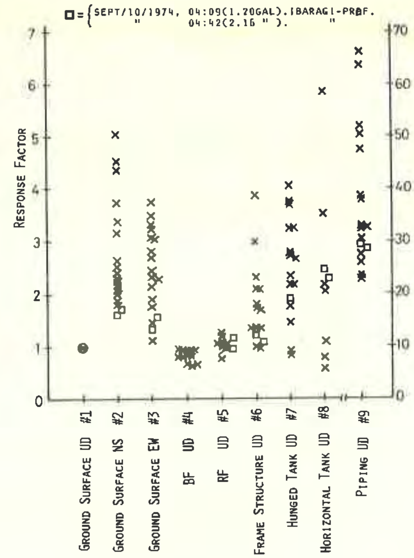

Fig. 5 : Distribution of Responses of Model Chemical Engineering Plant to Natural Earthquakes (Vertical Ground Motion)

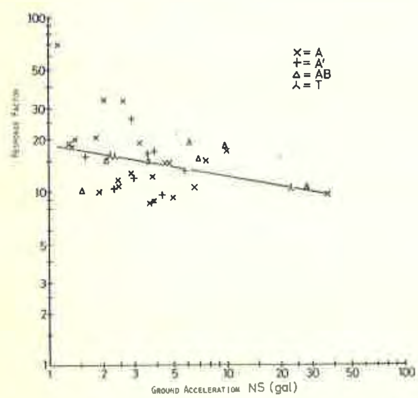

Fig. 6 : Response Factor *vs.* Ground Acceleration, Hanged Tank (to Near Field Earthquakes)

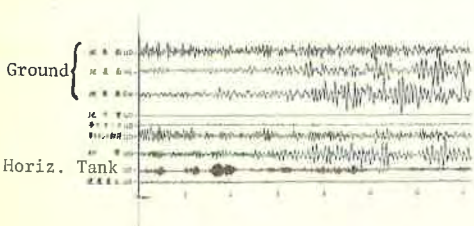

Fig. 8 : Record of Responses of Chemical Engineering Plant : Abnormal Response of Horizontal Tank

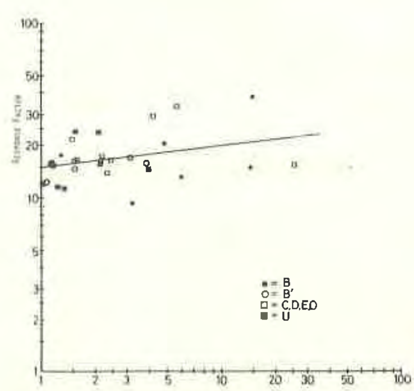

Fig. 7 : Response Factor *vs.* Ground Acceleration, Hanged Tank (to Other Earthquakes)

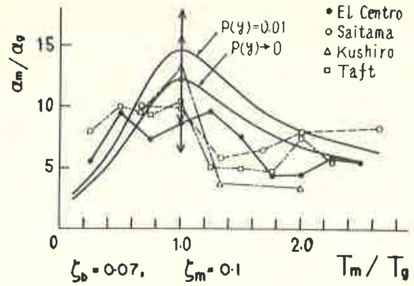

Fig. 9 : Response Factor of Two-degrees-of-freedom System : Theoretical and Simulated (*ref.* (12))

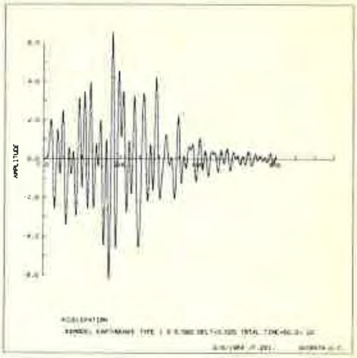

Fig. 10 : Simulated Ground Acceleration by Multi-degrees-of-freedom System Model

Critical Damping Ratio [%]	Relative Eigen Period of Structure T_b/T_g	Relative Dispersion [%]
7	0.2 1.0 4.0	12 21 29
2	0.2 1.0 4.0	12 32 44
0.7	0.2 1.0 4.0	18 42 50

Table 1 : Relative Dispersion Factor of a Single-degree-of-freedom System *vs.* Vibration Characteristics

	Eigen Freq. [Hz]	Critical Damping Ratio [%]
Piping UD	5.50	1
Horiz. Tank UD	18.3	1
" NS	6.38	1
" NS	4.72	2
Hanged Tank NS	4.73	3

Table 2 : Vibration Characteristics of Main Items

NS	Hanged Tank	Piping	Horiz. Tank	Tower Tank
Numb. of Data	60	58	59	59
Mean Resp. Fact.	16.04	22.86	5.87	1.344
Rel. Disp.	0.33	0.35	0.54	0.24
Max. Resp. Fact.	33.4	166	14.7	2.98

Table 3 : Statistical Characteristics of Responses to Horizontal Ground Motions

UD	BF	Hanged Tank	Piping	Horiz. Tank
Numb. of Data	12	16	17	9
Mean Resp. Fact.	0.83	2.49	37.8	2.31
Rel. Disp.	0.136	0.39	0.35	0.70
Max. Resp. Fact.	---	3.74	66.2	5.88

Table 4 : Statistical Characteristics of Responses to Vertical Ground Motions

	Average Acc. [gal]	Rel. Disp. [%]
Roof Floor	113.9	1
Lower Supporting of Piping	71.7	1
OD Point of Piping	229	9

Table 5 : Average Response and Relative Dispersion Factor of Model to Ten Identical Input Motion

Load Factor by Earthq.	Classification of Resp. Spec.	Rel. Disp. of Resp. d	$3d$ [%]
0.2	DRS	0.3	90
	FRS	0.5	150
0.5	DRS	0.3	90
	FRS	0.5	150

Mean + Resp. Fluct. + Error by Mag. Est. [%]	Over-stress [%]	Increase of Possibility of Fracture by Over-stress
240	28	1.4×10^{4}
300	40	1.4×10^{6}
240	70	3×10^{6}
300	100	1.3×10^{15}

Increase of Total Risk of Fracture
2×10^{0}
2×10^{2}
4.5×10^{6}
1.5×10^{11}

DRS: direct response spectrum
FRS: floor response spectrum

Table 6 : Increase of Possibility of Fracture under Over-stress Condition by Earthquake Loadings