

# SHAKE TABLE TESTS OF AN INDUSTRIAL BUILDING

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### SYNOPSIS

Shake table tests are conducted on a scaled frame model of an industrial building, which has irregular story heights, different beam span lengths, and uneven weight distribution. Its elastic and inelastic responses are observed under simulated earthquakes. The test results are used to verify the validity of a computer response analysis, which is based on the plastic hinge model of flexural members and a hysteresis rule for brace members. Additionally, the load effects derived from the test results are discussed comparing with the equivalent static load method in the design practice.

### INTRODUCTION

It is widely held as a design philosophy that ordinary buildings are allowed to undergo moderate inelastic deformation but they must avoid complete collapse under a severe earthquake. In past decades, with the availability of efficient computation tools, it becomes possible to predict inelastic earthquake response of a structure, once a mathematical model of its inelastic behaviour is formulated. Furthermore, large amounts of studies on elastic and inelastic responses are carried out and reflected on a procedure of earthquake resistant design.

In most of seismic provisions, however, an equivalent static load method is traditionally adopted instead of dynamic response simulations, because a design procedure based on computer response simulations is still so difficult to be accepted widely, except for the design of large-scale buildings or skyscrapers. For example, in the current seismic provisions attached to the Building Standard Law of Japan (revised in 1981), the effective intensity of seismic load effects is expressed in terms of the base shear coefficient by which the required shear strength of base story is determined. The provisions also define the vertical distribution of the required shear strength for multi-story buildings. This is specified by the distribution of the story shear coefficients, named as the  $A_i$ -coefficients, which are a function of vertical distribution of floor weights and the fundamental natural period, as shown in Fig. 1. These coefficients are told to be based on large amounts of response analyses on shear-type multi-story buildings.

An office-type structure is easily idealized into a single-line lumped-mass shear-type structure model where the floor weights are concentrated at the floor levels

and columns are considered to be springs connecting the lumped-masses. Industrial buildings, however, are generally different from the office building in structural framework. Sometimes the columns are irregularly placed and the story heights are not the same due to the arrangement of machines and equipment. In an extreme case, the concept of "story" can be hardly recognized in complicated structural frameworks. Moreover, the weights which must be carried by columns and beams are not neatly distributed. Therefore, the story shear coefficient recommended in the seismic provision, namely the  $A_i$ -coefficients, is sometimes inadequate to apply for the earthquake resistant design of such industrial buildings.

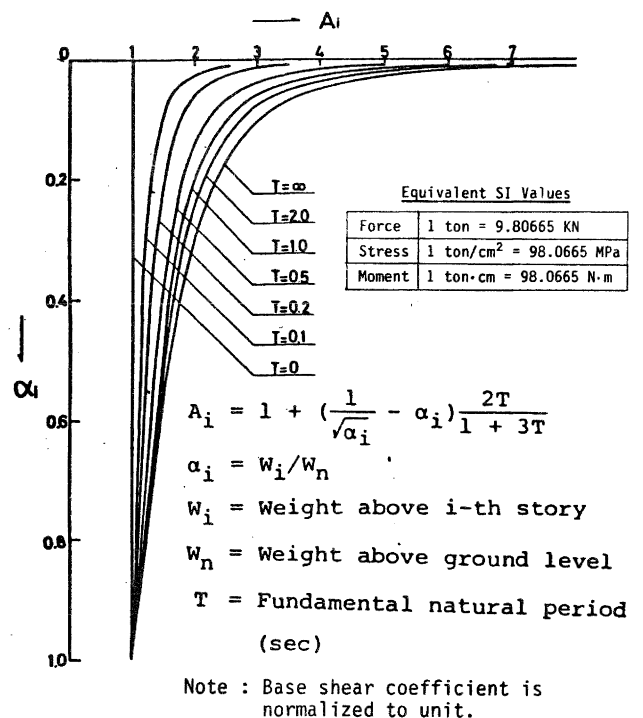


FIG. 1 - LATERAL SHEAR DISTRIBUTION FACTOR,  $A_i$ .

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According to the above mentioned reasons, dynamic response simulations are often carried out under several earthquake excitations, and the results are referred in the earthquake resistant design for these types of industrial buildings. A success or failure of the computer response simulation depends mainly on the validity of mathematical models that express the inelastic behaviour of framed structures. Therefore, experimental studies are indispensable in order to obtain accurate information not only about the inelastic behaviour of structural elements, but also about the global seismic responses of the whole structural system which are assembled from the structural elements.

In order to obtain experimental data about seismic responses of a complicated steel frame, shake table tests were conducted herein on a one-fifteenth scaled model of a turbine house in an electric power plant. Elastic and inelastic response tests under simulated earthquake ground motions were carried out as well as resonant response tests under harmonic excitations. The test results are used to verify a computer response analysis based on a simple formulation of hysteresis rule for the structural elements. In addition, the equivalent static loads are also discussed, which approximate the load effects derived from the dynamic test results.

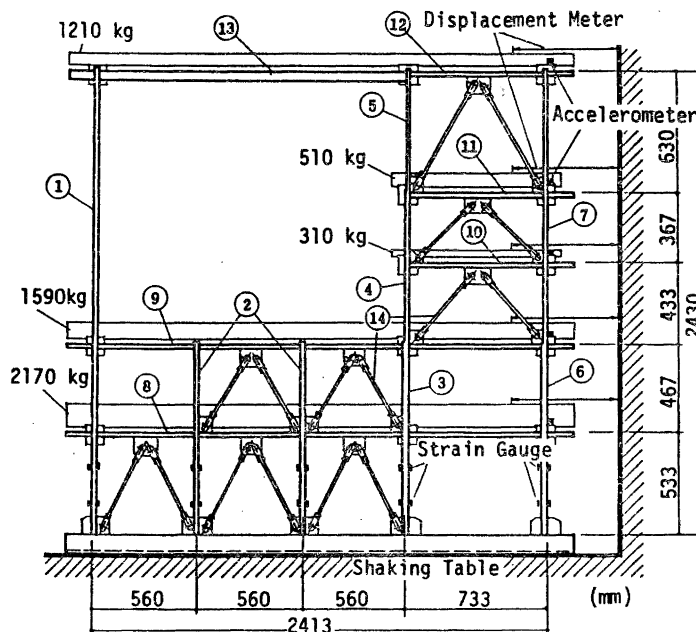
#### TEST FRAME AND INSTRUMENTATIONS

Shake table tests were carried out on a planar frame, which is a one-fifteenth scaled model of a turbine house building in an existing power plant. The elevation view of the test frame is shown in Fig. 2. The

column and beam members have rectangular sections,  $B \times H$ , which were cut out from the mild steel bars (JIS SS 41 grade steel having a nominal yield point of  $2.4 \text{ ton/cm}^2$ ).

The brace members were made of a seamless copper tube which were so selected that the strength ratio between the full plastic moment of the column and the product of the yield axial force times the length of the brace was equal to the ratio of the prototype. Under these conditions, if the scaled brace members had the same slenderness ratio as the prototype braces, they should have very thin-walled sections, which were difficult to fabricate. To avoid these difficulties we could not help setting the slenderness ratio of the scaled brace member larger than that of the prototype. Therefore, if the scaled braces were made of the same steel material as the columns and the beams, their buckling strengths became much smaller than their yield strengths, and they became a kind of tension-bar system, while there exists considerable contributions of the compressive strength in the prototype bracing system. This is the reason why we chose the copper material for the brace members, the yield strain of which is smaller than that of mild steel.

Two identical planar frames were placed on the shake table, and were connected at two and three column tops per floor by rectangular-section tie beams. At both ends of the tie beams, ball bearing devices were provided in order that the nodes of the test frame could rotate, in the plane of the framework, free from the influence of the tie beams. Bundles of steel sheets were



Members	B (mm)	H (mm)
①	19.0	22.1
②	18.9	8.92
③	19.0	27.1
④	19.0	24.1
⑤	19.0	13.9
⑥	19.0	22.1
⑦	19.0	15.1
⑧	9.00	18.9
⑨	9.01	20.0
⑩	9.05	24.1
⑪	9.05	24.1
⑫	9.03	23.1
⑬	9.10	46.0
⑭	φ 18 x 1.1	

FIG. 2 - ELEVATION VIEW OF TEST FRAME

added on the tie beams at the floor levels as floor weights.

The shake table device used during this study is installed at the Chiba Experiment Station, Institute of Industrial Science, University of Tokyo. It has a table of 3 metres square, and is driven by an electro-hydraulic servo mechanism. The maximum weight that the table can carry is 7.0 tons, and the horizontal table acceleration can reach twice the acceleration of gravity under maximum load. The shake table device can produce a two-dimensional movement, that is, not only a horizontal vibration but also a vertical one, but in this study it was used to provide the test frame with only a uni-directional horizontal vibration.

Accelerometers were installed at the floors and the shaking table to measure the absolute accelerations. From these acceleration data, the response inertial forces of the floors were directly calculated, and then the response story shear forces were obtained. The relative displacements of the floors to the shake table were also measured by the installed displacement meters. The wire strain gauges were adhered on the columns of the first story to obtain the bending moment of columns, and these data were used to calculate the base story shear carried by the columns. All of these data were simultaneously converted from analog to digital forms with the sample time of two milli-seconds, and recorded on a magnetic tape.

#### BRIEF DESCRIPTION OF TEST PROCEDURES

Preceding to the earthquake response tests, the resonant response tests were conducted. Under harmonic excitations with various frequencies, steady-state response tests were carried out carefully so that the test frame remained within elastic range. In the data processing of these tests, the FFT techniques were applied to remove the uninvited components other than the component of the aimed frequency (Shotaka et al. 1976). Magnification factors of response acceleration at each floor level are plotted to the frequency as resonance curves. These curves are fitted as shown in Fig. 3 by a

theoretical curve that is a function of modal properties, and the curves were used to identify the modal properties, such as mode shapes, natural frequencies, and modal damping factors. The results are summarized on Table 1, where the first three modes are shown. The modes higher than the third could not be recognized in these tests.

modes	f(Hz)	h(%)	floors	response amplitude	modal participation factor
1	6.82	2.3	2	5.73	0.261
			3	10.8	0.490
			4	16.2	0.739
			5	21.3	0.971
			R	32.8	1.49
2	12.2	1.1	2	28.2	0.625
			3	26.5	0.583
			4	12.1	0.266
			5	3.82	0.084
			R	-24.4	-0.536
3	16.8	1.1	2	2.42	-0.0533
			3	-3.75	0.0827
			4	-2.38	0.0524
			5	-2.23	0.0493
			R	1.38	-0.0306

TABLE 1 - RESULTS OF RESONANT RESPONSE TEST

An elastic and an inelastic earthquake response test were carried out. The acceleration wave form as the input ground motion at the tests was recorded at Hachinohe City in 1968. The peak value of original wave form was reduced to 100 cm/sec at the elastic response test and expanded to 1200 cm/sec at the inelastic test. The wave form was also compressed by one-half in the time domain (accelerated). The response acceleration spectra with the viscous damping ratios of 1.1% and 2.3% were shown in Fig. 4. In the inelastic response test, all the brace members buckled, and large inelastic deformations were observed at the first, the second, and the fifth story. Especially at the fifth story, the deformation angle due to story drift reached 0.05 radians, and the

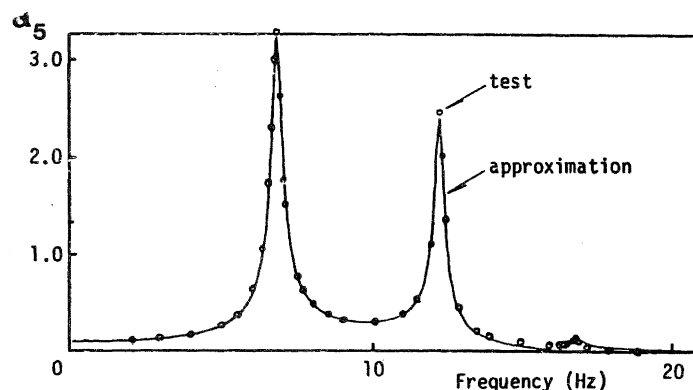


FIG. 3 - RESONANCE CURVE (ACCELERATION MAGNIFICATION FACTOR OF ROOF FLOOR,  $\alpha_5$ )

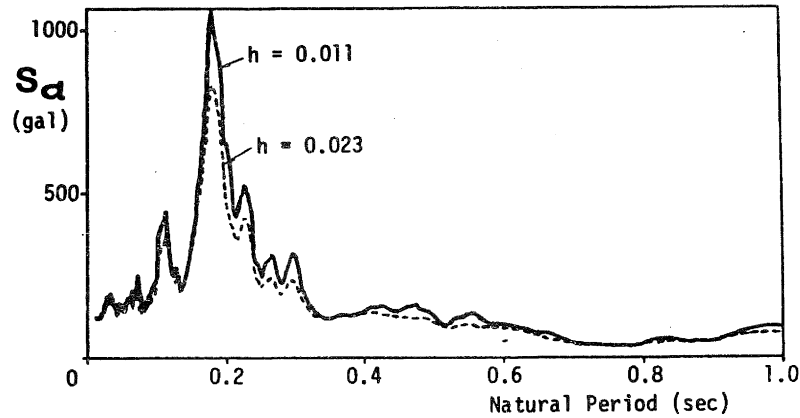


FIG. 4 - RESPONSE ACCELERATION SPECTRA,  $S_a$  FOR HACHINHOE EARTHQUAKE ( $A_{max}=100gal$ )

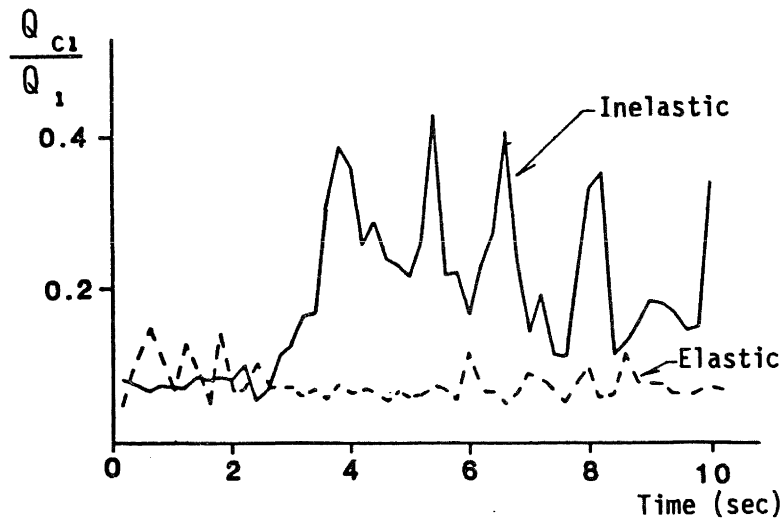


FIG. 5 - TIME HISTORIES OF 1ST STORY COLUMN SHEAR  $Q_{c1}$  NORMALIZED BY BASE SHEAR  $Q_1$

central portion of the roof beam was bent down inelastically by the braces.

The viscous damping effects in the test frame are so small that the contribution of the viscous damping forces can be ignored in the inertial forces observed at the floor level, and then, the total base shear force is obtained simply by summing up the inertial forces of all the floors. Shear force carried by the first story columns is calculated from the bending moments sensed by the wire strain gauges adhered on the columns. The participation ratio of the base story shear force transmitted by the columns was obtained as shown in Fig. 5. The participation ratio seems to be almost constant in the elastic case, for it is controlled only by the profile of the elastic stiffness among the members. On the other hand, the columns in the inelastic case have to carry large story shear forces after the brace members are buckled.

#### COMPARISON WITH COMPUTER SIMULATION

The fundamental problems in inelastic response analysis are how to formulate the mathematical models of restoring force characteristics of structural elements and how to assemble these element models into a whole vibrational system. In this study, elastic-inelastic behaviours of the columns, the beams, and the braces of the test frame are directly idealized into the hysteresis models proposed in the literatures. The computer program (I) used herein was developed for non-linear earthquake response

- (I) This computer program was developed as a part of the research project "Development of Earthquake Resistant Design of Turbine Buildings at Electric Power Plant" at the Building Center of Japan, and coded mainly by Mr. S. Hayase, Structural Engineer of Daiken Sekkei Co., Inc.

simulation of two dimensional frame structures, and called as "NOEL" in abbreviation. The "NOEL" program was designed so flexibly that users can make various choices about the modelling of an analysed frame. Main features of this program are briefly described as follows:

(1) The planar frame to be analysed can have an arbitrary shape of structural framework, so far as it is composed of straight single-line members. The members can be connected to each other at rigid or pinned nodes as well as with various type of springs. Inertial masses are idealized to be concentrated at the nodes.

(2) Hysteresis models of each member can be specified about three kinds of resultant forces at the member ends, that is, bending moment, shearing force, and axial force. These three hysteresis models per one member end are assumed to be independent from each other. Therefore, dynamic interaction effects among these resultant forces are ignored in this program. Of course, static interaction effects, such as the reduction of flexural strength due to constant axial loads, can be considered preceding to the dynamic analysis. As for the formulation of the hysteresis due to bending moment, the traditional plastic hinge concept is adopted. This model is so-called one-component model (Giberson, 1969), and a rigid-plastic rotational spring is inserted at a beam end when its yielding occurs.

(3) An equation of motion of a lumped-mass vibrational system is numerically integrated in a step-by-step manner using a central finite difference procedure, which is equivalent to the Newmark's  $\beta$  method with zero for  $\beta$ . When the numerical integration is completed up to the  $i$ -th step, the response displacements at the next  $(i+1)$ -th step can be calculated without iterations after the nodal restoring forces at the  $i$ -th step are evaluated. The member deformations, which are compatible with the nodal displacements, are traced by incremental analysis, and used to evaluate the elastic and inelastic restoring force of each member.

The modelling of the test frame was carried out in the following way:

(4) As for the columns and the beams, hysteresis models are considered only about the flexural deformations, while the shearing deformations are ignored and the axial ones are assumed to remain inelastic. The initial bending moment carried by plastic hinge is set to the full-plastic moment of the cross section. In the evaluation of the full-plastic moments, the yield stresses derived from the preliminary tension tests are used, and the interaction with the existing axial load due to floor weights are also considered. As for the hardening slope after yielding at the plastic hinge, 1.5% for the beams and 1.9% for the columns are assigned, respectively, of the elastic stiffness in the end moment vs. end rotation relationship under double curvature bending. These values are derived from the preliminary bending tests on simple beam specimens. The central portions of the beams which are

connected to brace members are strengthened by gusset plates, and assumed to be rigid in the analysis. Plastic hinges will be produced at both ends of this rigid zone, when this zone is strongly bent down by the braces after their buckling.

(5) As for the brace members, both ends are assumed to be pinned and the hysteresis loop in the axial force vs. elongation relationship is directly modelled by considering yielding, buckling, and post-buckling behaviour. A hysteresis model used herein was proposed by Shibata, Nakamura and Wakabayashi (1982), and it can express a gradually changed hysteresis curve as illustrated in Fig. 6. The critical stress  $\sigma_{cr}$  shown in Fig. 5 are formulated from the stable hysteresis loops in the cyclic loading tests, and then, its value becomes smaller than the Euler's buckling stress  $\sigma_e$ . Shibata et al. gave also an empirical formula to evaluate the critical stress  $\sigma_{cr}$  as a function of the yield stress  $\sigma_y$  and the Euler's buckling stress  $\sigma_e$ . As for the yield stress, the values derived from the preliminary tension tests on the brace members were used, and the buckling stress was calculated by assuming that the effective buckling length of the brace member is its clear length.

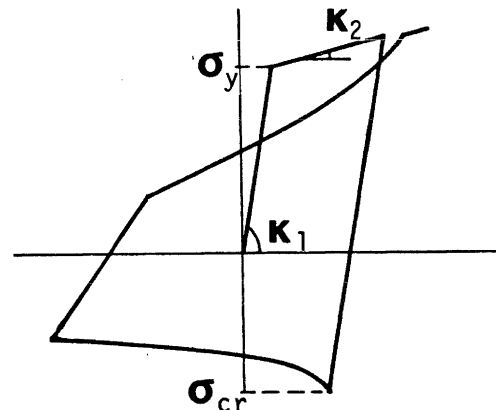


FIG. 6 - HYSTERESIS MODEL FOR BRACES

(6) Effects of secondary stress induced by floor weights after the deformation of the frame, so-called P- $\Delta$  effects, are considered in the analysis.

(7) In addition to the above mentioned hysteresis models, viscous damping effects were also considered. As for the damping constants, 2.3% of critical damping for the first mode and 1.5% for the higher modes were assigned after the resonance response tests.

Table acceleration records observed in the two earthquake response tests were used for the input ground accelerations in the response analysis. Peak response values obtained in the analysis, such as displacements, inertial forces, story drifts, and story shear forces, are compared with the test results in Table 2. Time histories of the response displacements at the first and the fifth floors in

(1) Elastic Response

(2) Inelastic Response

	story/floor	maxima		minima		maxima		minima	
		test	analysis	test	analysis	test	analysis	test	analysis
displacement at each floor (cm)	2	0.051 (3.91)	0.043 (3.91)	-0.044 (4.16)	-0.042 (4.16)	1.24 (4.19)	1.180 (4.21)	-0.36 (3.93)	-0.760 (4.89)
	3	0.081 (3.91)	0.081 (4.06)	-0.073 (4.16)	-0.079 (4.16)	2.41 (4.20)	2.506 (4.21)	-0.51 (3.95)	-1.463 (4.51)
	4	0.117 (4.08)	0.119 (4.07)	-0.103 (4.16)	-0.113 (4.15)	3.07 (4.19)	3.222 (4.21)	-0.71 (3.96)	-1.717 (4.51)
	5	0.160 (4.08)	0.148 (4.07)	-0.133 (4.16)	-0.139 (4.15)	3.49 (4.19)	3.585 (4.21)	-0.83 (3.96)	-1.865 (4.51)
	R	0.241 (4.08)	0.211 (4.08)	-0.191 (4.16)	-0.193 (4.15)	5.06 (5.37)	5.234 (4.20)	-1.97 (3.98)	-2.941 (4.97)
inertial force at each floor (ton)	2	0.185 (3.92)	0.219 (3.91)	-0.162 (3.98)	-0.177 (4.16)	1.24 (4.04)	1.255 (4.03)	-1.06 (4.46)	-1.099 (3.90)
	3	0.163 (3.92)	0.201 (3.91)	-0.154 (4.36)	-0.176 (4.16)	0.73 (4.20)	0.818 (3.84)	-0.76 (4.44)	-0.879 (3.91)
	4	0.035 (3.90)	0.039 (3.91)	-0.035 (4.16)	-0.037 (4.16)	0.20 (4.18)	0.284 (4.13)	-0.15 (4.42)	-0.300 (4.12)
	5	0.070 (4.08)	0.072 (4.07)	-0.069 (4.16)	-0.067 (4.15)	0.34 (4.17)	0.454 (3.83)	-0.33 (4.42)	-0.443 (3.76)
	R	0.247 (4.07)	0.233 (4.08)	-0.208 (4.34)	-0.212 (4.14)	0.60 (5.36)	0.513 (4.13)	-0.47 (3.98)	-0.392 (3.97)
Story drift (cm)	1	0.051 (3.91)	0.043 (3.91)	-0.044 (4.16)	-0.042 (4.16)	0.24 (3.67)	1.180 (4.21)	-0.36 (3.93)	-0.760 (4.89)
	2	0.039 (4.08)	0.039 (4.07)	-0.029 (4.16)	-0.037 (4.15)	1.17 (4.20)	1.330 (4.22)	-0.30 (3.96)	-0.745 (4.52)
	3	0.040 (4.08)	0.040 (4.08)	-0.030 (4.01)	-0.036 (4.14)	0.70 (4.23)	0.744 (4.23)	-0.22 (3.97)	-0.287 (5.81)
	4	0.047 (4.08)	0.030 (4.08)	-0.034 (4.35)	-0.027 (4.14)	0.46 (4.12)	0.390 (4.179)	-0.15 (3.98)	-0.183 (4.44)
	5	0.089 (4.08)	0.064 (4.08)	-0.062 (4.01)	-0.057 (4.14)	3.19 (6.55)	2.346 (6.57)	-1.40 (4.74)	-2.143 (4.99)
Story shear (ton)	1	0.592 (3.91)	0.640 (3.91)	-0.589 (4.16)	-0.623 (4.16)	2.11 (4.04)	1.899 (4.04)	-1.70 (4.47)	-1.757 (3.90)
	2	0.453 (4.08)	0.480 (4.07)	-0.436 (4.17)	-0.452 (4.15)	1.31 (4.18)	1.237 (4.07)	-1.10 (5.78)	-0.989 (3.95)
	3	0.346 (4.08)	0.338 (4.08)	-0.305 (4.16)	-0.310 (4.14)	0.82 (4.10)	0.797 (4.10)	-0.70 (4.35)	-0.608 (4.36)
	4	0.313 (4.08)	0.304 (4.08)	-0.269 (4.16)	-0.278 (4.14)	0.78 (4.12)	0.682 (4.14)	-0.59 (4.36)	-0.627 (4.38)
	5	0.247 (4.07)	0.233 (4.08)	-0.208 (4.34)	-0.212 (4.14)	0.60 (5.36)	0.513 (4.13)	-0.47 (3.98)	-0.392 (3.97)

Perthesized value is time in seconds

TABLE 2 - PEAK RESPONSE VALUES

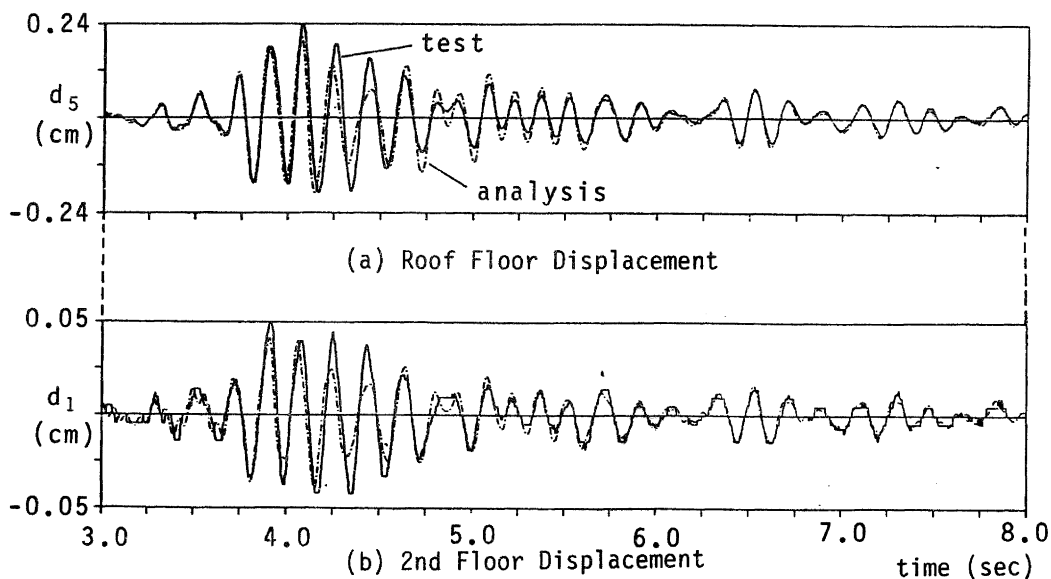


FIG. 7 - TIME HISTORIES OF RESPONSE DISPLACEMENTS (ELASTIC CASE)

the elastic case are shown in Fig. 7, where the analysed responses are shown by the solid curves. The analysed responses fairly well agree with the test results.

Fig. 8 shows hysteresis loops for the story vs. story drift relationship at the first and the fifth story in the inelastic case, and Fig. 9 shows their time

histories. In these figures, comparisons of the analysed and the test results are made in a similar manner in Fig. 7. Agreement between the analysed and the test results is not so good as in the elastic case, but it is found that the major trends in the inelastic responses can be traced by the presented analysis sufficiently for a practical use.

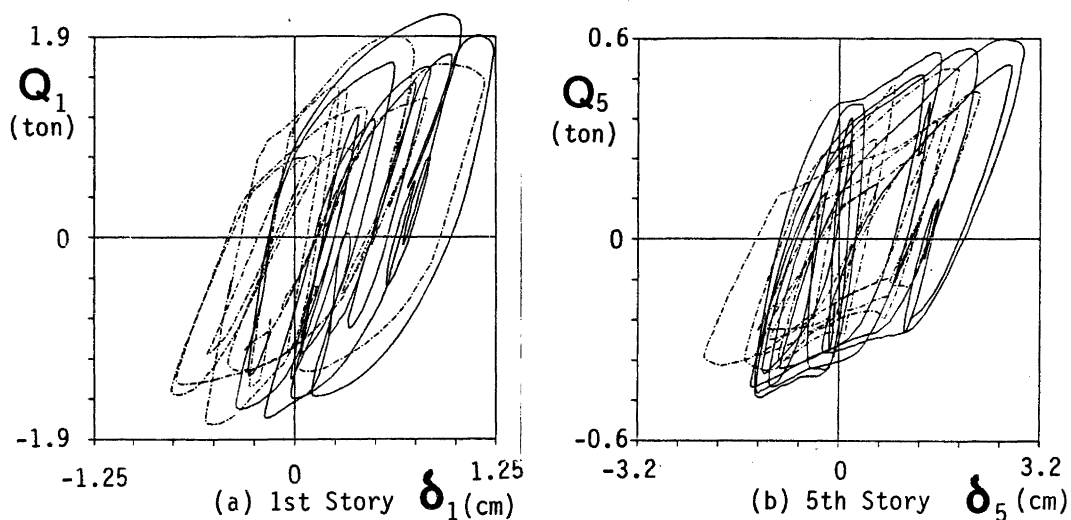


FIG. 8 - HYSTERESIS LOOP (INELASTIC RESPONSE)

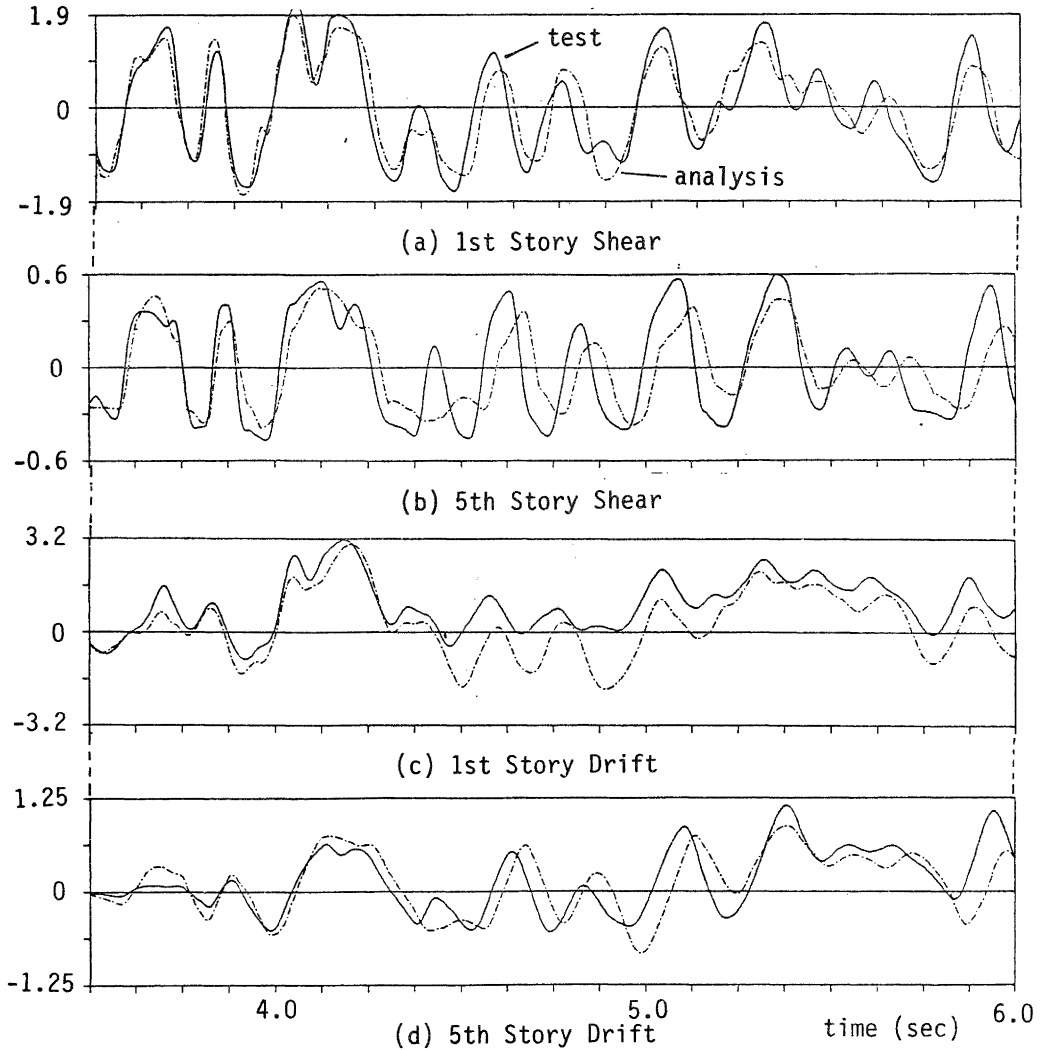


FIG. 9 - TIME HISTORY OF STORY SHEAR AND DRIFTS (INELASTIC RESPONSE)

LOAD EFFECTS DERIVED FROM TEST RESULTS

The maximum absolute values of response inertial forces at the floor levels during the elastic response test are shown in Fig. 10a. The peak values of story shear force in the same test are also shown in Fig. 10b. These values compared with the predicted ones by the modal analysis method. In the modal analysis method used herein, the maximum response values are approximated

by the square root of sum of the squares, often called "SRSS" in abbreviation, of the modal maxima which are obtained from the response spectra. Only the first three modes are considered in the SRSS method, where the fraction of critical damping for the response spectrum is assigned 1.1% for the first mode and 2.3% for the second and third modes, respectively. The prediction is fairly good as seen in Figs. 10a and 10b. It can be concluded that the SRSS method in

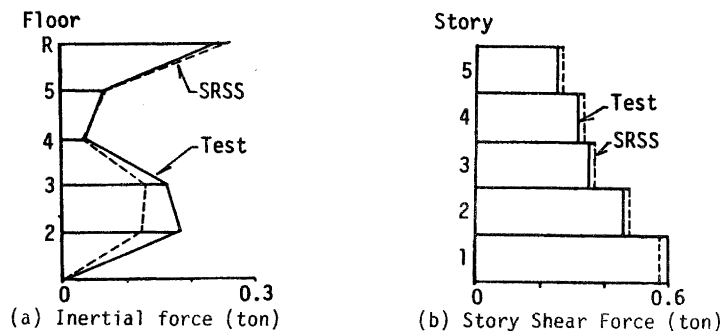


FIG. 10 - VERTICAL DISTRIBUTION OF INERTIAL FORCES AND STORY SHEARS (ELASTIC RESPONSE)

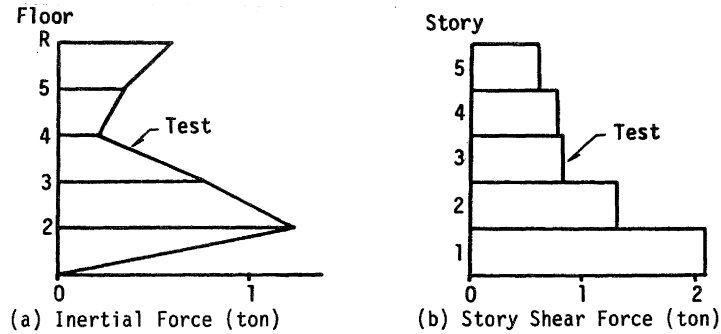


FIG. 11 - VERTICAL DISTRIBUTION OF INERTIAL FORCES AND STORY SHEAR FORCE (INELASTIC RESPONSE)

the modal analysis provided a reliable prediction in the elastic response behaviour, even though only a dominant few modes are taken into account.

Figs. 11a and 11b shows the maximum response values of the inertial forces and the story shear forces in the inelastic response test. The profiles of these values are different from the profile in Figs. 10a and 10b. The reason exists in the strength of the members.

Figs. 12a and 12b show the trajectory of the point in the  $Q_1 - Q_5$  plane, the coordinates of which represent the response story shear force  $Q_1$  at the first story and the response story shear force  $Q_5$  at the fifth story, respectively, during the elastic and inelastic earthquake response tests. The trajectory in Fig. 12a remains in the vicinity of the dashed line which represents the trajectory in the case of the first mode component for the elastic vibration. This evidence suggests that the load effect of the elastic response vibration can be expressed by a quasi-static and proportionally increasing load.

In the plastic analyses the assumption of "proportional loading" makes it considerably simple to calculate the ultimate strength of a structure. Indeed, we must calculate the ultimate strength under all

combinations of seismic loads at floors without this assumption. Therefore, when we can find the most likely existing quasi-static load mode among response vibration modes, the strength of the frame under earthquake excitation can be examined by a static analysis.

When the above mentioned equivalent static load mode is given, it is theoretically possible to proportion the member sizes so as to yield at the same time as an optimum design. This design is not realized, however, in practical design procedure, because member sizes are not available for exactly suitable selection. Fig. 12b shows the response results of such a case. The trajectory in the figure seems to be suppressed along the  $Q_5$  axis due to the insufficient strength of the fifth story in comparison with the sufficient strength of the first story. This unbalance in strength causes the suppression and the scattering of the trajectory. This scattering causes the induction of other vibration modes than the first mode which is very dominant in the elastic response. Therefore, if considerable amounts of inelastic behaviour are expected, the design seismic load should be determined accounting the influence of a coupling of modes other than the initial dominant mode. This can be considered by emphasizing the participation of higher modes in the SRSS manipulation.

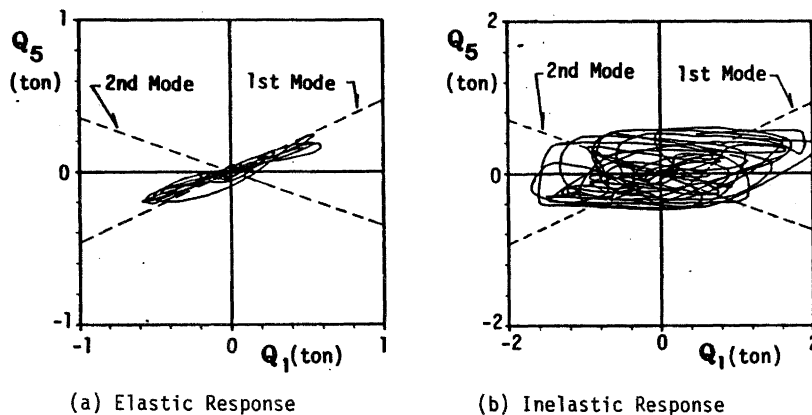


FIG. 12 - TRAJECTORIES OF STORY SHEAR FORCES (1ST STORY AND 5TH STORY)

CONCLUDING REMARKS

A scaled model of a five-story braced steel frame, which had a complicated shape of framework and an uneven weight distribution, was tested on the shake table. Its elastic and inelastic responses to simulated earthquakes were observed and compared with the computer analyses. From these tests and analyses, the following conclusions can be derived:

(1) The participation among the braces and other moment resistant members to carry seismic forces is often used to characterize a braced frame. It was observed in the tests that this construction was changed during the responses. In the elastic tests this participation was kept constant and controlled by the stiffness profile of the members, while it showed unsteady changes with the post-buckling restoring forces of the braces in the inelastic tests.

(2) A frame response analysis described herein, which was based on the plastic hinge concept for the moment-resistant members and a hysteresis model for the brace members, could fairly well trace the major trends of the inelastic responses observed in the tests. One reason for this success seems that the information of the preliminary tests on the members were well reflected on the modelling of the member behaviours.

(3) Seismic forces in the elastic response can be evaluated with the SRSS of the modal maxima at the response acceleration spectrum. This was confirmed by the test results. Moreover, an equivalent static load method can be applied to find which failure mode will most possibly occur, for a dominant load mode usually exists as observed in the elastic test.

(4) When a braced frame undergoes considerable amounts of inelastic deformations, the above mentioned dominancy of a load mode is diminished, as observed in the inelastic tests. In spite of this phenomenon, the load mode adopted in the design for moderate earthquakes can be used to find the most possible failure mode also in the design for ultimate earthquakes, because a frame with an unbalanced strength profile will be considerably damaged in the initial failure mode before the appearance of another failure mode. For example, the fact that the test frame was damaged most of the fifth story can be easily found by the static loading proportional to the first mode. It is another problem, however, to determine a profile of seismic forces or damage potentials induced by earthquakes. We recommend the changes of vibration features after yielding should be taken into account to determine the profile of the design seismic forces. This can be achieved by taking the higher mode response maxima more intensively into the SRSS manipulation.

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