RELIABILITY OF EXISTING BUILDINGS IN EARTHQUAKE ZONES - FINAL REPORT

by

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1. INTRODUCTION

1.1 General Remarks

In his introduction to the Selected Papers of Hardy Cross which was published by the University of Illinois Press in 1963, Professor Nathan M. Newmark made the following recollection:

> "I shall always remember the occasion of my oral prelims when Cross asked a question, the details of which I do not now recall, but probably about the choice of a design criterion, which seemed to me to be so controversial I could not think of an answer. The only reply I could think of was, 'I don't know!' Slowly that ineffable grin spread across his face; he replied, 'That's the correct answer. It may well be the most correct answer you've given today. But don't be discouraged. Neither does anyone else know.'"

Although this incident was not directly related to the problem of structural reliability, it does serve the purpose of pointing out that a well-considered reply "I don't know" can be the correct and appropriate answer to some controversial and complex questions. The safety evaluation of existing structures is such a complex problem, to which a completely satisfactory solution is not available to-date.

In October 1945, the late Professor Alfred M. Freudenthal published a paper entitled "The Safety of Structures" with the objective "to analyze the safety factor in engineering structures in order to establish a rational method of evaluating its magnitude". This historical paper marked the beginning of structural reliability studies and was selected for inclusion with many discussions in the 1947 Transactions of the American Society of Civil Engineers [1].

During these past three decades, much progress has been made in the subject area of structural safety and reliability. The state-of-the-art as of 1972 was summarized in a report of the American Society of Civil Engineers Structural Division Task Committee on Structural Safety [2]. Subsequently, members of the same Task Committee presented a reliability-based design code format to the civil engineering profession [3]. Meanwhile, the theory of structural reliability has been applied to solve safety-related problems in earthquake, wind, ocean, aerospace, and nuclear engineering [4]. Recently, the reliability-based load and resistance factor design (LRFD) for steel buildings is considered to be the prototype for the next generation of structural design codes in the States [5-12].

To-date, most studies on structural reliability analysis have been concerned with the calculation of failure probabilities for given mathematical representations of idealized structural systems and enviornmental conditions.

Even for simplified and idealized systems, relatively few special cases of the structural reliability problem have been solved mathematically in closed form. In spite of the many recent improvements and refinements in structural analysis, it is still difficult to obtain adequate mathematical representations of the overall structural behavior in the nonlinear range corresponding to various stages of severe structural damage. Furthermore, certain significant factors in structural reliability cannot be evaluated in an objective manner. In summary, available results of theoretical and analytical developments in structural reliability cannot be easily applied for the safety evaluation of complex existing structures.

1.2 Objective and Scope

The objectives of this research project are (a) to formulate reliability criteria, and (b) to analyze available test data in terms of damage probability and other probabilistic response measures. Some three technical reports, 6 conference papers, and two journal articles were supported in part through this research grant. In the following, results of these studies are summarized and reviewed. Recommendations for further research are also presented.

2. LITERATURE REVIEW

2.1 System Identification in Structural Dynamics

System identification refers to techniques which are developed for obtaining a mathematical representation of a specific physical system when both the input to the system and the corresponding output are known [13,14]. In general, a system identification technique consists of the following three parts: (a) the determination of the form of the model with certain system parameters, e.g., a set of second-order ordinary differential equations for the description of the dynamic behavior of a given mechanical system; (b) the selection of a criterion function using some means of the "goodness of fit" from the model response to the measured response, when both the mathematical model and the actual system are subjected to the same input; and (c) the selection of an algorithm for the modification of system parameters so that the discrepancies between the behavior of the mathematical model and that of the actual system can be minimized.

During this past decade, structural response records with or without known forcing functions have been collected and analyzed to obtain better mathematical representations of the dynamical behavior of existing structures [15-21]. During the first phase of this investigation, Chen et. al. [18-19] critically reviewed more than 40 references and tabulated their pertinent information concerning the application of system identification techniques in structural engineering. The linear lumped-parameter models are found to be the most widely used ones because of their simplicity. Common methods of analysis include the use of modal expansion and transfer function, and various least-squares estimation techniques. For nonlinear models, invariant imbedding and dynamic programming filters, least squares filters, quasilinear methods, extended Kalman filters, and maximum likelihood methods have been used.

By necessity, dynamic testing of structures for the purpose of performing system identification must be conducted at small response amplitudies so that the serviceability and safety limit states are not reached during these tests. Consequently, the effectiveness of the resulting mathematical model is restricted to the linear or slightly nonlinear range of the structural behavior.

Natural hazards such as strong-motion earthquakes have caused severe damage to existing structures, and the safety evaluation of structures under such extreme loading conditions is very important indeed. With the "realistic" mathematical models resulting from system identification studies, it is possible to simulate the structural response to such extreme loads and thus to evaluate the serviceability and safety of the strucutre under consideration. However, there exists the paradox that (a) the applicability of most "realistic" models of the structure is limited to small-amplitude and linear response range, (b) the catastrophic loading conditions are likely to cause the structures to respond beyond the linear or "near-linear" behavior which is usually assumed, and (c) the severe loadings may cause serious damages in the structure and thus change the structural behavior appreciably from those in the mathematical model resulting from system identification studies.

It is important that the extent of damage in structures can be assessed following each major catastrophic event or at regular intervals for the evaluation of aging and decaying effects. On the basis of such damage assessment, appropriate decisions can be made as to whether a structure can and should be repaired [22,23].

2.2 Damage Assessment of Existing Structures

The damage of a given structure can be studied both experimentally and analytically in case of need [24,25]. Experimental studies include either field surveys or laboratory tests. Field surveys include the determination of exact locations of failed components and other evidence of distress, the application of various non-destructive testing techniques to the remaining structure, the discovery of poor workmanship and construction details, and proof-load and other load testing of a portion of a very large structure. Meanwhile, samples can be collected from the field and tested in the laboratory for strength and other mechanical and structural properties. Analytical studies frequently consist of the examination of the original design calculations and drawings, the review of project specifications, the performance of additional structural analyses incorporating field observations and test data, and the possible explanation and description of the event under consideration. The state-of-the-art for damage assessment of existing structures has been reviewed recently [26,27]. Although such general procedures are known to exist, the detailed methodology, especially the decision-making process, remain as priviledged information for a relatively few experts. Such priviledged information and specialized knowledge are being transmitted to younger engineers primarily through many years of working experience and the development of engineering "intuition" and professional "judgement", which are highly personal and subjective in nature.

2.3 Structural Identification

The concept of structural identification has been discussed at various stages of development since 1973 [15,16,26,27]. Structural engineers are mainly interested in identifying the damage and reliability functions, in addition to obtaining the equations of motion. On the other hand, the updated equation of motion using test data and system identification can be a tool for the estimation of expected damage and reliability of existing structures in the future.

When a structure is inspected for the purpose of making damage assessment, a series of tests may be conducted and the resulting data can be analyzed accordingly. Quantities which can be measured and recorded in testing structures include the load, the deformation (or strain), and the acceleration. From these experimental measurements, mechanical properties such as stiffness and strength and dynamic characteristics such as natural frequency and damping can be estimated. In addition, indications of damage such as cracks and local buckling in the plastic range can be detected visually by experienced inspectors. As an example, binoculars have been used by persons looking for color change in window panes in a certain tall building which indicate the presence of flaws causing the eventual breakage of window glasses. For metal structures which are subjected to repeated load applications, dye-check, ultrasonic or x-ray devices may be used to find and measure small and hidden fatigue cracks which indicate structural damage.

When a structure undergoes various degrees of damage, certain characteristics have been found to change. In testing a reinforced concrete shear wall under reversed loading conditions, free vibration tests were performed to estimate the fundamental natural frequency and damping ratios [28]. Results of these tests as given by Wang, Bertero, and Popov [28] indicate that (a) the frequency decreased monotonically with damage while the damping ratio increased initially and then decreased, and (b) the repaired specimen was not restored to the original condition as indicated by free-vibration test data. Similar results were reported by Hudson [17], Hilgardo and Clough [29], and Aristizabal-Ochoa and Sozen [30], among others.

Recently, comprehensive experimental results of dynamic full-scale tests were obtained for a multi-story building structure [31] and a 3-span highway bridge [32]. Galambos and Mayes [31] tested a rectangular 11-story reinforced concrete tower structure, which was designed in 1953, built in 1958, and tested in 1976. The large-amplitude (and damaging) motions were induced with the sinusoidal horizontal movements of a 60-kip lead-mass which was placed on hardened steel balls on the eleventh floor. This lead-mass can be displaced up to ± 20 inches and the frequency capacity was 5Hz with the use of a servo-controlled hydraulic actuator, one end of which is fastened to the building frame. The maximum horizontal force range was ± 30,000, pounds. These test results indicate that the natural frequency decreased with increasing damage in general. Similarly, Baldwin et al [32] concluded from their testing of a three-span continuous composite bridge that changes in the bridge stiffness and vibration signatures can be used as indicators of structural damage under repeated loads.

3. FORMULATION OF RELIABILITY EVALUATION

3.1 A Damage Function

One possible approach to the structural identification problem is to obtain a damage function, the parameters of which are then to be estimated using testing and inspection data. Blume and Monroe [33] assumed that the damage is linearly related to ductility factor with "0" denoting elastic behavior and "1" denoting collapse. That is,

$$\text{Damage} = \frac{\langle Z-y\rangle^1}{c-y} \tag{1}$$

where Z = maximum displacement response

y = yield displacement

c = displacement at collapse

$\langle x\rangle^n$ = singularity function such that, for $n \geq 0$, $\langle x\rangle^n = 0$ when $x < 0$, and $\langle x\rangle^n = x^n$ when $x \geq 0$.

Bertero and Bresler [34] stated that (a) the lateral displacement ductility factors generally provide a good indication of structural damage, and (b) the interstory drift is a more important factor in causing nonstructural damage. Bresler [35] discussed the relative merits of using plasticity ratio (residual deformation to yield deformation) and the ductility. For structures which are subjected to cyclic plastic deformations with decreasing resistance, the ratio of the initial to j^{th}-cycle resistance at the same cyclic peak deformation was also suggested.

For monotonic loading conditions, Oliveira [36] defined a damage ratio function, DRF, as follows:

$$DRF = \left[\frac{<Z - y>}{c - y}\right]^{b} \tag{2}$$

where b is a material and structural paramenter. It is interesting to note that Equation 2 is analogous to a damage function which was developed for axially-loaded mild steel speciman subjected to low-cycle and high-amplitude reversed plastic deformations [37]. This earlier damage function as given by Yao and Munse [37] was used to evaluate the damageability of structures by Kasiraj and Yao [38] for a given earthquake excitation, and later by Tang and Yao [39] for random ground motions. In an unpublished technical note, Rosenblueth and Yao [40] used the following damage function in their pilot study of cumulative damage of seismic structures:

$$D = \sum_{i=1}^{n} a_i <\frac{Z_i}{y} - 1>^{b_i} \tag{3}$$

The reasoning for the form as given in Equation 3 is presented in the Technical Note in Appendix A (see Equation AI-4). Unfortunately, currently available test data are still insufficient to either validate the form of such a damage function or to estimate these parameters for reinforced concrete structures.

3.2 Application of Pattern Recognition

Fu and Yao [41] considered the problem of damage assessment in terms of pattern recognition. In the theory of pattern recognition [42,43], data are collected from a physical system such as an existing building structure with the use of transducers. These transducers may include (a) human eyes with which the size, number, and location of cracks can be measured and recorded, and (b) accelerometers with which ground motion and structural response can be obtained. A pattern space and a reduced-pattern (feature) space are then extracted.

Finally, a decision function or classifier is applied to obtain the classification, which in our case is the damage state as desired.

As an example of reliability evaluation of structures, Kobo [44] analyzed some 30 bridges which were damaged to various degrees during the Kanto earthquake in 1923, the Kukui earthquake in 1948, and Niigata earthquake in 1964. Among these 30 bridges, 5 collapsed, 9 were severely damaged, and 16 were not heavily damaged. A total of nine items were used to characterize each bridge including (a) ground condition, (b) liquefaction, (c) type of structure, (d) type of bearing, (e) maximum height of abutment or bridge pier, (f) number of spans, (g) width of substructures crest, (h) seismic intensity in MMI, and (i) foundation. In each item, there were two or three categories for further classification. The degree of damage, A_i, is assigned to i^{th} bridge with 1.5 to 5 for those five collapsed bridges and 0.8 to 2 for all others. A multiplicative relationship of weighting factors, w_{jk} (for k^{th} category of j^{th} item) was used. For these 30 bridges, the weighting factors are obtained and a criterion was proposed for the evaluation of seismic strength of existing bridges.

Recently, Fenves [45] discussed the potential application of artificial intelligence and pattern recognition to structural design and detailing. The following three levels were discussed: (a) the first level is to develope strategies for processing existing design specifications, (b) the second level is to make specifications more closely related to engineers' processing needs and limitations, and (c) the third level is to provide explicit means for discrimination, evaluation, and selection.

3.3 Application of Fuzzy Sets

According to Zadeh [46, 47], as the complexity of a system increases, our ability of making precise and yet significant statements concerning its behavior diminishes. Consequently, the closer one looks at a real-world problem, which is usually complex, the fuzzier its solution becomes. The application of fuzzy sets to several civil engineering problems was reviewed recently [48,49]. Most civil engineering structures are indeed complex systems, the behavior of which can not be easily and clearly described at present.

In a recent technical report [49], fundamental elements of the theory of fuzzy sets as given by Zadeh [47] and Kaufmann [50] are summarized along with several structural engineering examples from Yao [48] and a simplified version of an example on structural reliability from Brown [51]. An attempt was then made to begin the application of the theory of fuzzy sets to the complex problem of damage assessment of existing structures.

4. ANALYSIS OF AVAILABLE TEST DATA

Analytical results as presented by Chen et al [52,53] are summarized herein. In this study, two simple methods are presented for the identification of linear parameters as a function of time during a strong-motion earthquake. Method I is used to find parameters ω_n and ξ from two simultaneous equations which result from the linear and first-mode equation of motion at time t and time t + Δt by using measured earthquake and response data. Assuming the time interval Δt is taken such that the structural behavior remains linear, parameters at any time t can be found as follows:

$$\omega_n^2(t) = \frac{(\ddot{x}_{01}+\ddot{y}_1)\dot{y}_2 - (\ddot{x}_{02}+\ddot{y}_2)\dot{y}_1}{y_1\dot{y}_2 - y_2\dot{y}_1} \qquad (4)$$

and

$$\xi(t) = - \frac{\ddot{x}_{01}+\ddot{y}_1+\omega_n^2(t)y_1}{2\omega_n(t)\dot{y}} \qquad (5)$$

where $\ddot{x}$, y, $\dot{y}$, and $\ddot{y}$ denote respectively measured ground acceleration, first-mode displacement, velocity, and acceleration response of the structure; and the subscripts "1" and "2" refer to the evaluation these time functions at times t, and t+Δt, respectively. The solution can be improved by selecting Δt when variance of ω_n and/or variance of ξ are small by taking the mean and variance of ω_n and ξ at different Δt.

In method II, the natural frequency, ω_n, and damping ratio, ξ, are found by using the least squares-fit. These two parameters are estimated by minimizing the integral-squared difference, E, between the excitations of the structure and the linear model. Hence,

$$E = \sum_{i=n_j}^{n_k} (\ddot{x}_{oi} - \ddot{x}_{oi}^{(1)})^2 = \sum_{n_j}^{n_k} (\ddot{x}_{oi} + \ddot{y}_i + 2\xi\omega_n \dot{y}_i + \omega_n^2 y_i)^2 \tag{6}$$

$$\omega_n^2 = \frac{(\Sigma\ddot{y}_i\dot{y}_i + \Sigma\ddot{x}_{oi}\dot{y}_i)\Sigma\dot{y}_i y_i - (\Sigma\ddot{y}_i y_i + \Sigma\ddot{x}_{oi} y_i)\Sigma\dot{y}_i^2}{\Sigma\dot{y}_i^2 \Sigma y_i^2 - (\Sigma\dot{y}_i y_i)^2} \tag{7}$$

and

$$\xi = - \frac{\Sigma\ddot{y}_i y_i + \Sigma\ddot{x}_{oi} y_i + \omega_n^2 \Sigma y_i^2}{2\omega_n \Sigma\dot{y}_i y_i} \tag{8}$$

where intervals (n_j, n_k) are segments of the records.

The Union Bank is a 42-story steel frame structure, which sustained minor (nonstructural) damage such as plaster cracking and tile damage during the 1971 San Fernando earthquake. Its rough estimate of earthquake repairs was $50,000 out of an initial construction cost of $30,000,000 [54]. The distribution of the natural frequency obtain from Method I is shown in Table 1. The value from 1.25 rps to 1.35 rps is the mode. As given in Table 2, the natural frequency is found to decrease segment by segment except for the last value obtained from least squares fit. The loss of stiffness as indicated by this change in natural frequency seems to be the result of cracking and other types of degradation of nonstructural elements during the occurence of large-amplitue earthquake response.

Table 1. Analysis of Union Bank Data - Distribution of Natural Frequency Obtained from Method I [53].

ω_n	< 0.945	0.95 to 1.05	1.05 to 1.15	1.15 to 1.25	1.25 to 1.35	1.35 to 1.45	1.45 to 1.55	1.55 to 1.65	1.65 to 1.75	1.75 to 1.85	1.85 to 1.95	1.95 to 2.05	> 2.05
No.	26	23	16	29	36	29	30	17	11	4	6	6	27

Table 2. Comparison of Analytical Results of Union Bank Data [53].

Method I (t & t+1.2 sec)			Method II		Modal Minimization Method by Beck [55]	
Time Interval	ω_n	ξ	ω_n	ξ	Time Interval	ω_n
0-14			1.44	0.15	5-15	1.42
14-28	1.32	0.08	1.34	0.09	10-20	1.37
28-42	1.30	0.08	1.25	0.08	15-25	1.35
42-56	1.25	0.07	1.31	0.11	20-30	1.33

The Union Bank building was strongly excited by a large pulse in the ground motion at approximately 10 seconds. The natural frequency was found to be very irregular during the first 10 seconds, when the denominator in Equation 1 is near zero. Therefore the calculated natural frequencies of first segment are not accurate and should be ignored. Consequently, the damping ratio, ξ, is also inaccurate due to such irregular values of ω_n. Otherwise, the properties at large amplitudes of displacement are more relevant for the application of these methods by comparing the displacement record and results of parameters. In addition, very small values of time segments should be avoided because of the presence of measurement noise and errors. Otherwise, these proposed methods are found to be useful and efficient to determine the parameters for linear structures and can be used to detect the presence and occurrence of nonlinear behavior in an existing structure. In Table 2, the results of using these methods show agreement with those of modal minimization method by Beck [55].

5. DISCUSSION AND RECOMMENDATIONS

5.1 Discussion of Research Results

Ideally, the behavior of any physical system in given environmental conditions can be described with mathematical expressions. In reality, however, it is difficult to obtain such a detailed and accurate mathematical representation for the damagement assessment and reliability evaluation of existing structures because of the following reasons:

(a) Although the linear and slightly nonlinear behavior of structures can be successfully analyzed mathematically or numerically, the analysis of structures through various stages of damaging loading conditions remains to be a challenging problem [31].

(b) Most existing civil engineering structures consist of extremely complex systems. The classification and identification of damage states for such complex systems require further studies.

(c) There exist many factors which cannot be evaluated objectively in the damage assessment and reliability evaluation of existing structures.

In the current practice, relatively few highly-experienced and well-trained structural engineers can successfully perform damage assessment and reliability evaluation for existing structures. Even when these experts are willing, their specialized knowledge cannot be transmitted to younger engineers in a direct and systematical manner without many years of working together. The ultimate goal of this research project is to develope a more direct and systematical methodology so that more engineers can be trained in a shorter time period than it is required at present.

With this long-term objective in mind, several approaches were examined following extensive library research into various aspects of the subject area [18, 19, 20, 26, 27]. An attempt was made to obtain a damage function [40]. To seek a rational framework within which the problem can be formulated, the theory of pattern recognition was considered [41]. In addition, an exploratory study of the theory of fuzzy sets was made for possible applications in finding decision functions [49]. Preliminary results of analyzing available test data are also presented [52,53].

5.2 Recommendations for Further Research

While it is desirable to continue various research efforts in structural studies by other ivestigators, the following alternative approaches are recommended.

5.2.1 Search for a Damage Function

The pilot study as given in Appendix A should be continued. Although available experimental data were not intended for the establishment of such a damage function, attempts may be made to develope a method analyzing the voluminous test data for such a purpose. Moreover, experimental investigators should be encouraged to consider the possible development of a damage function in the design and conduct of their future testing programs.

5.2.2 Formulation of the Problem of Damage Assessment:

Efforts should be continued to examine the problem of damage assessment in the contest of pattern recognition. It is desirable to obtain a mathematical formulation with emphases on (a) the formulation of a decision function or classifier and (b) the development of a procedure for the feature extractor. The usefulness of currently available field data for damage assessment should be examined. Moreover, the need for additional information not currently available should be considered.

5.2.3 Identification of Significant Parameters:

With the mathematical formulation of the problem, attempts should be made to identify significant parameters, which are measurable or can be computed from currently available data. In addition, it is desirable to find additional parameters of importance which are not being collected and analyzed at present.

It is expected that these patterns can be ranked according to their respective significance in relating to the overall damage state.

5.2.4 Illustration and Demonstration:

Special cases of experimental results and field data should be analyzed for the purpose of illustration and demonstration of such a methodology. In so doing, the practicality and feasibility of this methodology can also be examined.

5.2.5 Collection, Processing, and Interpretation of Existing and Available Test Data and Reocrds:

Field records and laboratory test data should be collected. Whenever it is possible, the methodology thus developed should be applied. Additional tests can be proposed if such information and data are not readily available for the implementation of this methodology.

5.2.6 Interaction with Practicing Engineers:

Because of the practical nature of the ultimate goal of this research topic, close contact must be maintained among academic researchers and practicing engineers. In fact, several experts in practice have cooperated with the investigator to-date. Moreover, they promised to continuously provide their valuable advice and future cooperation in case of need.

5.2.7 Control of Structural Damage:

A review paper on the topic of structural control was presented at the 1979 IUTAM Symposium on Structural Control [56]. The written manuscript is attached to this final report as Appendix B. As the application of active control becomes more feasible and practical in the structural engineering profession, it is desirable to study further the application of active control for safety considerations.

5.2.8 Development of Inspection and Testing Procedures

Eventually, results of these above mentioned studies can be utilized in the development of inspection and testing procedure for various types of structures. Such an inspection and testing procedure can be used to assist engineers in deciding whether a new structure is acceptable as well as whether a damaged structure can be repaired.

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APPENDIX A:

Technical Note

ON SIESMIC DAMAGE AND STRUCTURAL RELIABILITY

by

E. Rosenblueth[1] and J. T. P. Yao[2]

INTRODUCTION

During these past several decades, most building structures have been designed to behave elastically during moderately strong earthquakes. For economical and other practical reasons, some plastic deformations (and thus permanent damage) are expected to occur during extremely large earthquakes [A1]. The damage resulting from repeated plastic deformations can be cumulative, and certain structural failures were related to low-cycle fatigue phenomenon [A2].

Much progress has been made since the theory of structural reliability was introduced by Freudenthal in 1947 [A3]. Although some attempts were made [A4], the effect of seismic damage on structural reliability has not been clearly defined.

The objective of this technical note is to formulate the effect of earthquake-induced damage on structural reliability. A simple example is included for the purpose of illustration. Moreover, possible modifications to include the effects of weathering, wear, and corrosion are also considered.

BASIC FORMULATION

Define the seismic reliability function as the probability that the structure will survive each and every strong-motion earthquake

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during a given time interval [o,t], i.e.,

$$R(t) = P\left[\bigcap_{i=o}^{N(t)} S_i \quad , \quad \bigcup_{n=o}^{\infty} (N(t) = n)\right]$$

$$= \sum_{n=o}^{\infty} P\left[\bigcap_{i=o}^{N(t)} S_i \mid N(t) = n\right] \quad P[N(t) = n]$$

$$= \sum_{n=o}^{\infty} P\left[\bigcap_{i=o}^{n} S_i\right] P[N(t) = n] \tag{A1}$$

where

S_i = event that the structure survives the i^{th} earthquake

$N(t)$ = random process denoting the number of strong-motion earthquakes occurring during the time interval [o,t].

Using the discrete hazard function

$$h(i) = P\left[S_i^{\ c} \mid \bigcap_{k=o}^{i-1} S_k\right] \tag{A2}$$

where the superscript "c" denotes the complementary operation of the event, i.e., $S_i^{\ c}$ denotes the event that the structure does not survive the i^{th} earthquake, Eq. 1 can be expressed as follows:

$$R(t) = \sum_{n=o}^{\infty} \left[\prod_{i=o}^{n} (1 - h(i))\right] P[N(t) = n] \tag{A3}$$

AN ILLUSTRATION

To illustrate for the evaluation of the seismic reliability as given in Eq. A3, let Y_i be a random variable denoting the structural response (demand) to i^{th} earthquake, and X_i be a random variable representing the resistance (capacity) of the structure to the demand resulting from the i^{th} earthquake.

It is reasonable to assume that,

$$X_i = (1 - \sum_{k=o}^{i-1} D_k)\, x_o \tag{A4}$$

where D_k denotes the structural damage resulting from k^{th} earthquake. For simplicity and without the loss of generality, the initial resistance, x_o, is taken as being a determinstic quantity herein. Assuming statistical independence between Y_i and $D(i-1) = \sum_{k=o}^{i-1} D_k$, it can be shown that

$$h(i) = \frac{\int_o^1 f_{D(i-1)}(x)[1 - F_{Y_i}((1-x)x_o)]dx}{\int_o^1 f_{D(i-1)}(x)dx} \tag{A5}$$

where f denotes the probability density function, and F denotes the cumulative probability distribution function.

As it is described in Appendix I, it seems to be reasonable to use an exponential distribution as an approximation for the random variable D_k, i.e.,

$$f_{D_k}(x) \simeq ae^{-ax} \quad , \; x \geq o \tag{A6}$$

If we assume that

$$f_{Y_i}(y) = ce^{-cy} \quad , \; c > \frac{1}{x_o} \; , \; y \geq o \tag{A7}$$

we obtain, for this simple example,

$$h(i) = e^{-cx_o} \left(\frac{a}{a - cx_o} \right)^{i-1} \frac{1 - \sum_{k=o}^{n-2} \frac{(a - cx_o)^k}{k!} e^{-(a - cx_o)}}{1 - \sum_{k=o}^{n-2} \frac{a^k}{k!} e^{-a}} \tag{A8}$$

Note that h(i) is an increasing function as expected. R(t) can now be computed using Eqs. A3 and A8. It can be easily shown that R(t) is a decreasing function of t.

EFFECT OF WEATHERING, WEAR AND CORROSION

Let t_1 denote the time for structure to deteriorate completely due to weathering, wear, and corrosion, and $T^{(i)}$ be a random variable denoting the waiting time for the occurrence of i^{th} earthquake during [o,t], then

$$X_i = \left(1 - \sum_{k=o}^{i-1} D_k\right) \left(g\left(\frac{T^{(i)}}{t_1}\right)\right) x_o \tag{A9}$$

where $g\left(\frac{T^{(i)}}{t_1}\right)$ is a decreasing function of the argument. For examples,

$$g\left(\frac{T^{(i)}}{t_1}\right) = 1 - \left(\frac{T^{(i)}}{t_1}\right)^w \tag{A10}$$

or

$$g\left(\frac{T^{(i)}}{t_1}\right) = \left(1 - \frac{T^{(i)}}{t_1}\right)^w \tag{A11}$$

Correspondingly, the hazard function can be modified as follows:

$$h(i) = \frac{\int_0^t dt \int_0^1 dx \; f_{D(i-1)}(x) \; f_{T^{(i)}} \{1 - F_{Y_i}[(1 - x) g(\frac{t}{t_1}) x_o]\}}{\int_0^t dt \int_0^1 dx \; f_{D(i-1)}(x) \; f_{T^{(i)}}(t)} \tag{A12}$$

The structural reliability function R(t) can now be computed by using Eqs. A2 and A12 accordingly.

DISCUSSION

At present, there does not seem to exist any usable cumulative damage law for complete structures. A hypothesis based on plastic deformations is suggested in Appendix I for the purpose of discussion. There exists a need to establish such failure criteria for various types of structures.

In summary, a rational approach to relating seismic damage and structural reliability is proposed herein. This framework can be further developed to become more practical in the future when laboratory and other necessary data become available.

ACKNOWLEDGMENT

This work was performed jointly by E. Rosenblueth and J. Yao during J. Yao's visit to the Instituto de Ingenieria in 1977. The support of this visit by the Instituto, Purdue University, and National Science Foundation are gratefully acknowledged.

APPENDIX AI DISTRIBUTION OF SEISMIC DAMAGE

Consider a structure, which is designed to possess a ductility capacity of μx_y, where x_y is the yield deformation for a design earthquake with magnitude m_y at distance ℓ_y. Using the relationship [A1],

$$x_y = H(\ell_y)e^{\delta m_y} \tag{AI-1}$$

The peak demand in plastic deformation due to i^{th} earthquake with magnitude M_i at distance L_i is given by

$$X_{p_i} = \langle X_i - x_y\rangle = \langle H(L_i)e^{\delta M_i} - H(\ell_y)e^{\delta m_y}\rangle \tag{AI-2}$$

where the square brackets denote the singularity function such that $\langle s\rangle = s$, for $s \geq 0$, and $\langle s\rangle = 0$ for $s < 0$. On the basis of a low-cycle fatigue criterion which was developed for small metal specimens [A6], it is postulated that the incremental damage, D_i, resulting from i^{th} earthquake is given by

$$D_i = \sum_{j=1}^{n_i} \left(\frac{X_{p_{ij}}}{\mu x_y - x_y}\right)^{\beta_i} \tag{AI-3}$$

where $X_{p_{ij}} = j^{th}$ application of the plastic deformation at the critical section during the i^{th} earthquake. For ease in computation, let $X_{p_i} = \max_j (X_{p_{ij}})$, we have

$$D_i \leq n_i \left(\frac{X_{p_i}}{\mu x_y - x_y}\right)^{\beta_i} = \alpha_i \langle Y_i - 1\rangle^{\beta_i} \tag{AI-4}$$

where $$Y_i = \frac{H(L_i)}{H(\ell_y)} e^{\delta(M_i - m_y)}$$

and $$\alpha_i = \frac{n_i}{(\mu - 1)^{\beta_i}}$$

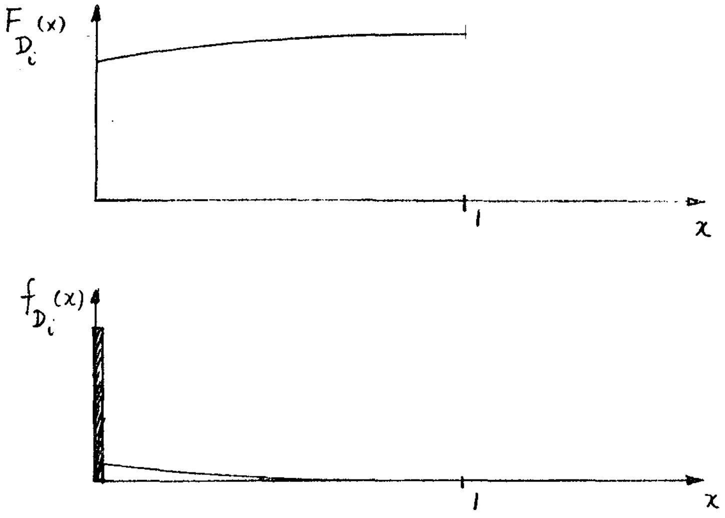
It can be shown that

$$F_{D_i}(x) = F_{Y_i}\left[1 + \left(\frac{x}{\alpha_i}\right)^{1/\beta_i}\right] \qquad \text{(AI-5)}$$

and

$$f_{D_i}(x) = \frac{1}{\alpha_i \beta_i}\left(\frac{x}{\alpha_i}\right)^{1/\beta_i - 1} f_{Y_i}\left[1 + \left(\frac{x}{\alpha_i}\right)^{1/\beta_i}\right] + F_{Y_i}(1)\delta(x) \qquad \text{(AI-6)}$$

The fact that most structures are designed to behave elastically implies that $F_{Y_i}(1)$ is very close to unity. For such cases, Eqs. AI-4 and AI-5 are sketched as shown in Figure I-1.



It is believed that an exponential distribution function can be used to approximate Eqs. AI-4 and AI-5, i.e.,

$$F_{D_i}(x) \simeq 1 - e^{-ax} , \qquad \text{(AI-7)}$$

and

$$f_{D_i}(x) \simeq ae^{-ax} \qquad \text{(AI-8)}$$

where a is large and $x \geq 0$. When the form of $F_{Y_i}(y)$ is available or given, the approximate value of a can be established for appropriate ranges of x-values.

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APPENDIX B:

IDENTIFICATION AND CONTROL OF STRUCTURAL DAMAGE*

by

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INTRODUCTION

When I was a college student, I learned to always choose a larger section and thus a more conservative alternative in structural design whenever such an option existed. The reasons were that, for civil engineering structures, (a) the weight of the structure was not a critical constraint, and (b) the cost of construction material was not a major consideration. Moreover, design codes are traditionally conservative. It is well known that most civil engineering structures to-date are relatively massive, stiff, and stable. Nevertheless, over the years, a few of these conservatively designed structures have been damaged due to unusually severe natural hazards, defective materials, as well as human errors in design and construction. Recently, more flexible structures are being designed and built because (a) more sophisticated methods of analysis are available, (b) cost of material is becoming a more significant design factor than ever before, and (c) taller (or longer) and thus more flexible structures are being attempted for architectural and other considerations.

In an ideal situation where the disturbance to be encountered and the resistance of the structure are completely known, it is relatively simple to design a comfortable and safe structure. In reality, there always exist uncertainties in predicting future loading conditions as well as in estimating

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* General Lecture, presented at the IUTAM Symposium on Structural Control, University of Waterloo, Waterloo, Ontario, Canada, 4-8 June 1979.

structural resistance. Moreover, there exist discrepancies between the actual structural behavior and its corresponding mathematical representations used in the process of structural analysis and design. To-date, several motion-controlling devices have been used or porposed for comfort and/or safety considerations. The development of structural control has been reviewed recently [B1], and the state of the art is presented in detail by various experts during this Symposium. Nevertheless, it is still difficult to design and construct a structure even with the effective use of control systems, which will completely avoid the possibility of being damaged during its intended lifetime.

The objective of this paper is to stimulate discussion of possible topics in structural control for further research and development. The literature on structural control is briefly summarized from a structural engineer's viewpoint. The general applicability of structural control under various conditions is reviewed. Finally, the identification of human and/or structural response state in existing structures and its effect on structural control are discussed.

LITERATURE REVIEW

The history of control theory relating to structural applications was reviewed by Zuk [B2]. An attempt is made herein to briefly summarize the recent development of structural control.

About ten years ago, I thought that the application of active control could solve most difficult problems in structural engineering. Ideally, flexible structures such as extremely tall buildings or long bridges can be designed to resist only the operational gravity loads and the active control system can take care of any side-sway motions resulting from lateral load effects. Such being the case, we can bypass such problems as statistical uncertainty in predicting future loading conditions and complicated structural analysis. Of course, I learned quickly that not only the control theory is a well-established and difficult subject in itself, but also its application to structural engineering requires further investigations. Instead of providing a simple overall solution, the application of control theory has helped to create many new challenging problems in structural engineering.

In 1968, Zuk [B3] discussed the concept of kinetic structures, an example of which was the application of tendon control as proposed independently by Freyssinet in 1960 and by Zetlin in 1965. Meanwhile, Wright [B4] and Nordell [B5] suggested the use of active systems, which can be used to resist any exceptionally high overloading of a given structure. Later, the use of initially slack cables in forming bilinear hardening structures which are subjected to earthquake loads was studied [B6,B7]. The concept of structural control was presented to the structural engineering profession in 1971 [B8]. As an example, the use of thruster engines to generate impulsive control forces was mentioned. Meanwhile, Gaus [B9] suggested that it is desirable to search for an optimum combination of passive and active control devices.

In 1970, Wirsching [B10,B11] studied the use of passive motion-reducing devices and suggested several means for the improvement of structural safety under earthquake loading conditions. The displacement response of one-, five-, and ten-story building structures to strong-motion earthquakes was simulated with the use of an analog computer. The Gumbel Type I distribution of maxima was used in the statistical description of these peak response data. Results of this study showed that the isolator system was the most effective one among the five passive control systems thus studied.

In 1967, Masri B12 studied the possible application of "two-particle" impact dampers. Gupta and Chandrasekaran [B13] investigated the use of an absorber system for the reduction of earthquake effects. It was also mentioned that gyroscopes were being studied for use in the torsional stabilization of suspension bridges in Japan [B14]. Nevertheless, most studies in this direction in recent years are concentrated on the practical implementation of isolator systems, including the use of energy-absorbing devices (through plastic deformation), and tuned mass dampers.

Green [B15] suggested in 1935 to construct buildings with a flexible first story to obtain favorable response to earthquake excitations. Fintel and Kahn [B16] reported that buildings without shear walls on the first floor suffered less damage than those with shear walls during the 1963 Skopje and the 1964 Caracas earthquakes. Therefore, it was suggested that buildings can be designed with a first story which is stiff enough to resist wind loads but flexible enough to isolate the upper floors from seismic effects.

In 1969, Matsushita and Izumi [B17] proposed the use of non-circular rollers which would cause the building to rise when it is displaced laterally. The weight of the building would then act as a restoring force, which depends on the shape of these rollers.

During the 1970 Gediz, Turkey earthquake, it was reported by Penzien and Hanson [B18] that stretching of anchor bolts at column bases of several buildings prevented the occurrence of more serious structural damage than those actually occurred. In 1973, Kelly, Skinner, and Heine [B19] tested three types of energy-absorbing devices, which undergo plastic torsion and thus absorb the kinetic energy in the structure due to earthquake motions. The use of such devices as isolators in seismic structures was explored by Skinner, Beck and Bycroft [B20]. Recently, the behavior of two types of mild steel energy-absorbing devices was given by Kelly and Tsztoo [B21], who showed that these devices have substantial hysteretic energy absorbing capacity. Kelly and Tsztoo [B22] also presented results of earthquake simulation tests of model frames with such energy-absorbing devices. These resulsts indicated the feasibility of using such devices for aseismic design [B23].

In 1976, Skinner, Bycroft, and McVerry [B24] studied the combined use of the energy-absorbing devices and laminated rubber bearings which possess adequate horizontal flexibility for the isolation of nuclear power plants during earthquakes. They concluded that a high reliability can be achieved for base-isolator components on the basis of extensive laboratory tests. Tyler [B25] reported on results of dynamic shear tests on such laminated rubber bearings, and concluded that these bearings are suitable for use as base isolators. Robinson and Tucker [B26] studied a lead-rubber isolator consisting of a steel-reinforced elastomeric bearing with a lead insert fitted in its center. They also recommended for its use in base-isolation systems for the protection of structures during earthquakes. Eidinger and Kelly [B27] demonstrated experimentally the possibility of using such bearings as isolators. Jolivet and Richli [B28] reported on the application of similar reinforced-elastomer/friction-plate bearing systems in the foundation design of nuclear power plant in South Africa. Recently, a massive research program to study the use of steel energy absorbing restrainers and their incorporation into nuclear power plants was described in a summary report [B29]. In a companion volume, the current uses of energy absorbing devices were reviewed [B30]. The optimal design of an earthquake isolation system was also investigated [B31].

Crandall et al [B32,B33] studied the slip of friction-controlled mass under earthquake loading conditions. Recently, Nemat-Nasser [B34] is making an analytical study of the vibration of a continuous viscoelastic slab resting on viscoelastic support for such practical applications.

Klein et al [B35] studied the use of shutter-like appendages to stabilize wind induced vibrations in tall buildings. This concept is being extended and experimental studies are now in progress [B36,B37].

A tuned mass damper was installed on the 59th floor of the 914-foot Citicorp building to minimize the discomfort experienced by occupants on windy days. The device weighs 400 tons with two spring damping mechanisms and a control system which is used to collect data and controls the motion of this mass [B38,B39]. Although additional steel plates were welded to bolted connections later [B40], the retrofit was said to be unrelated to the effectiveness of the tuned mass damper which is working extremely well. This interesting topic was discussed by Petersen [B41] recently.

In a technical report, Yao and Tang [B42] discussed the application of an active control system using impulsive control forces. For a single-degree-of-freedom system, the control force was chosen as follows:

$$F = -\sum_{i=1}^{\ell} a_i \, H(|X| - \xi_i) \, \text{sgn}\,(X) \, H(X \cdot \dot{X})$$

where a_i denotes the force magnitude increment at the ith control level,

ξ_i denotes the specified displacement of the ith control level

H(.) denotes the Heaviside unit-step function

sgn(.) denotes the signum function.

Similar control forces were given for a two-story building structure. Results of numerical examples indicated that such impulsive control forces can be effective in reducing the displacement responses of one- and two-story structural frames to the 1940 El Centro earthquake excitation. For the purpose of illustration, the effectiveness of such control laws in reducing the displacement response of a two-story building structure is shown in Figure 1.

Yang and Yao [B43] explored the application of the classical stochastic control theory to civil engineering structures. Yang [B44] showed that significant reduction in covariances of structural responses to stationary wind forces as well as nonstationary earthquake effects can be obtained with the use of optimal control. It is realized, however, that this type of controller cannot be used to avoid any given peak responses.

Sae-Ung et al [B45,B46] used the Monte-Carlo method to study structural control for comfort purposes. A comfort control law using impulsive control forces was applied to a 40-story building structure, and these simulation results indicated that such a control law is feasible in terms of energy requirements.

Martin and Soong [B47] applied modal control in changing specific dynamic modes and system stiffness directly. A design procedure for modal control has also been developed by Chang and Soong [B48]. Recently, various types of tendon control have been studied by Roorda [B49], Schorn [B50], Yang and Giannopoulos [B51], and Abdel-Rohman and Leipholz [B52,B53]. In a different type of application it was proposed to apply the control theory in the development of design codes [B54].

APPLICABILITY OF STRUCTURAL CONTROL

Recently, structural control for the purpose of maintaining human comfort has been successfully implemented in practice [B39-B41]. Meanwhile, many questions on practicality remain in the application of structural control for safety purposes. Some even say that practicing engineers will never accept the idea of using active control systems to ensure structural safety. In this section, an attempt will be made to assess the applicability of structural control under various circumstances.

It may be desirable to review the following several facts. First, it is difficult to predict future loading conditions with limited amount of past records. Secondly, it is not economically feasible to continue the tradition of designing and building massive and stiff civil engineering structures. Last but not least, uncertainties exist in material resistance, mathematical representations, methods of structural analysis, and various human factors. In the case of structures with control systems, the reliability of various components of the control system cannot be overlooked. Depending on the failure consequence, a structure can be designed to possess a certain level of safety. Nevertheless, it is not practical to design and build a structure even with control systems which can completely avoid any damage during its intended life-time.

For the sake of discussion, consider a given structure with and without structural control as shown schematically in Figure 2. Let $X = X(t)$ be a vector of random processes denoting various loading conditions. Furthermore, let S_o denote structural responses without the control system and S_i denote responses of the ith design of the structure with control system, $i=1,2,\ldots,n$. Define the following damage states:

D_{oi} = "little or no" damage for ith design
$= (S_i < \ell_e)$

D_{1i} = "Tolerable" damage for ith design
$= (\ell_e \leq S_i < \ell_t)$

D_{2i} = "Repairable" damage for ith design
$= (\ell_t \leq S_i < \ell_r)$

D_{3i} = "Severe" damage for ith design
$= (S_i \geq \ell_r)$

where ℓ_e, ℓ_t, and ℓ_r denotes elastic limits, tolerable damage limits, and repairable damage limits, respectively. If the cost for all these designs are comparable, the design to be chosen can be the one with max $P(D_{oi})$, $i = 0,1,\ldots,n$. The computation of $P(D_{oi})$ can be complicated enough even for structures without control systems. Although the reliability problem related to structural control was explored in 1972 [B55], practical and complete solutions are not available to-date.

A point of view concerning the applicability of structural control is summarized in Figure 3. Consider two types of control system, namely passive and active systems. The passive system such as dampers and isolators are always available and operational. On the other hand, consider three types of active control systems, say one each involving small, moderate, and large control forces. The active control system with small control forces seems to be suitable for comfort control and for keeping the structure within the null-damage state D_o. The small control force can also be associated with those loads which exceed the design values but still are lower than the exceptional ones. The active control system with moderate control forces may be appropriate for maintaining the structure within tolerable or repairable damage limits. The moderate control force can also be associated with those loads which range between exceptional and abnormal loads. The large control force should not be used until the structure is near collapse when it is subjected to abnormal loading conditions. The adjective "almost never" implies a very small or zero probability of occurrence.

The argument against structural control for safety considerations will certainly continue. With this brief discussion, it is hoped to call attention to the fact that (a) it is difficult to predict with great certainty the extraordinary or abnormal loading conditions which may occur during the intended lifetime of the structure, and (b) different magnitudes and types of control force may be used under various loading conditions. If and when the failure consequence of a particular structure is extremely grave, the use of one or more levels of active control systems may be warranted.

STRUCTURAL IDENTIFICATION

To effectively control the motions of a structure, it is necessary to be able to describe the characteristics of the particular structure. Currently available mathematical representations result from generalizations of existing knowledge in the structural engineering profession. Following the completion of the construction process, each civil engineering structure possesses its own characteristics, the precise description of which is difficult to obtain with the use of any general mathematical model. In recent years, techniques of system identification [B56,B57] have been applied to structural engineering [B58]. At present, the study of structural identification includes mathematical modelling, damage assessment, and reliability of existing structures on the basis of field observations and test data [B59]. A comprehensive summary and discussion of the subject matter was presented recently [B60].

The interrelationship between structural control and structural identification is shown schematically in Figure 4. Basically, a set of warning limits can be established such that the active control devices are activated whenever one or more of these warning limits are exceeded. Structural identification techniques can be applied to detect any damage and to decide whether such damage is tolerable. If and when permanent and moderate damage occur, the structure must undergo a detailed inspection. If it is found necessary, the structure must stop functioning and major repairs should be implemented. When the damage is found to be severe, the structure must then be demolished and rebuilt.

It is noted that such terms as "tolerable" and "repairable" damage are not clearly defined for an existing structure, which is usually a very complex system. Therefore, it is difficult to identify such limit states in reality. As a possible approach to the solution of this problem, the application of pattern recognition is being explored [B61].

CONCLUDING REMARKS

It is encouraging to note that there has been an increasing interest in research activities concerning structural control during this past decade. With the cooperation of experts from various disciplines including structural engineering, theoretical and applied mechanics, and control theory, more significant contributions to this subject area can be expected in the near future.

One of the most challenging problems seems to be the one relating mathematical representations to the structural behavior in the real world. I believe that experimental results such as those reported by Roorda [B62] will be most promising in bridging this gap. Nevertheless, there remains the problem of obtaining precise yet significant solutions of the behavior of complex systems. As it was stated by Zadeh [B63], our ability of making precise and yet still significant statements concerning the system behavior diminishes with increasing complexity of the system. Consequently, the closer one looks at a complex real-world problem, the fuzzier its solution becomes.

In summary, much progress has been made in the subject area of structural control. Nevertheless, there remain many challenging problems. The most significant contribution of this Symposium was to establish worthwhile communication among various relavant disciplines. I expect that various experts can collaborate to solve these and other problems successfully in the foreseeable future.

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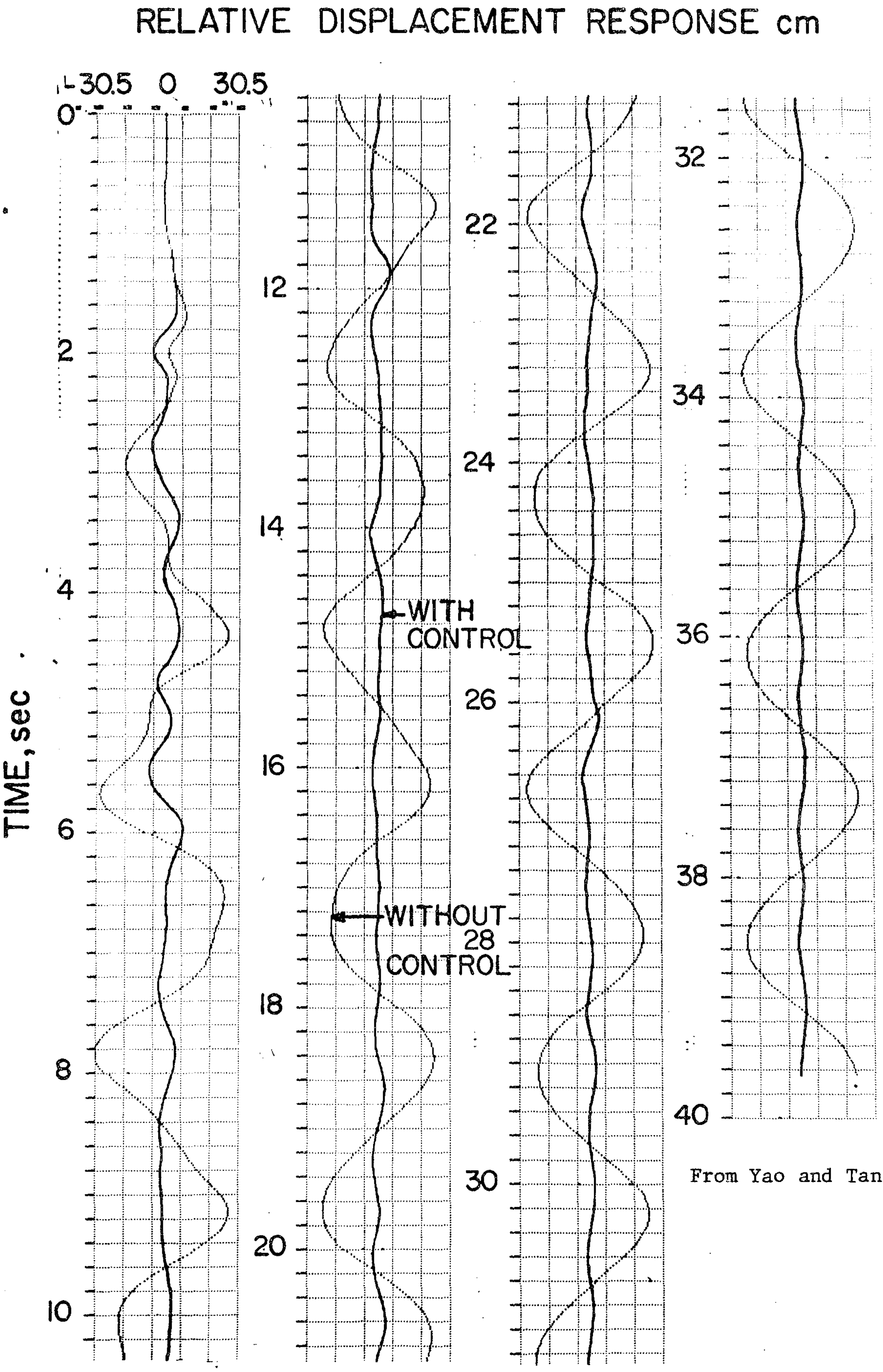
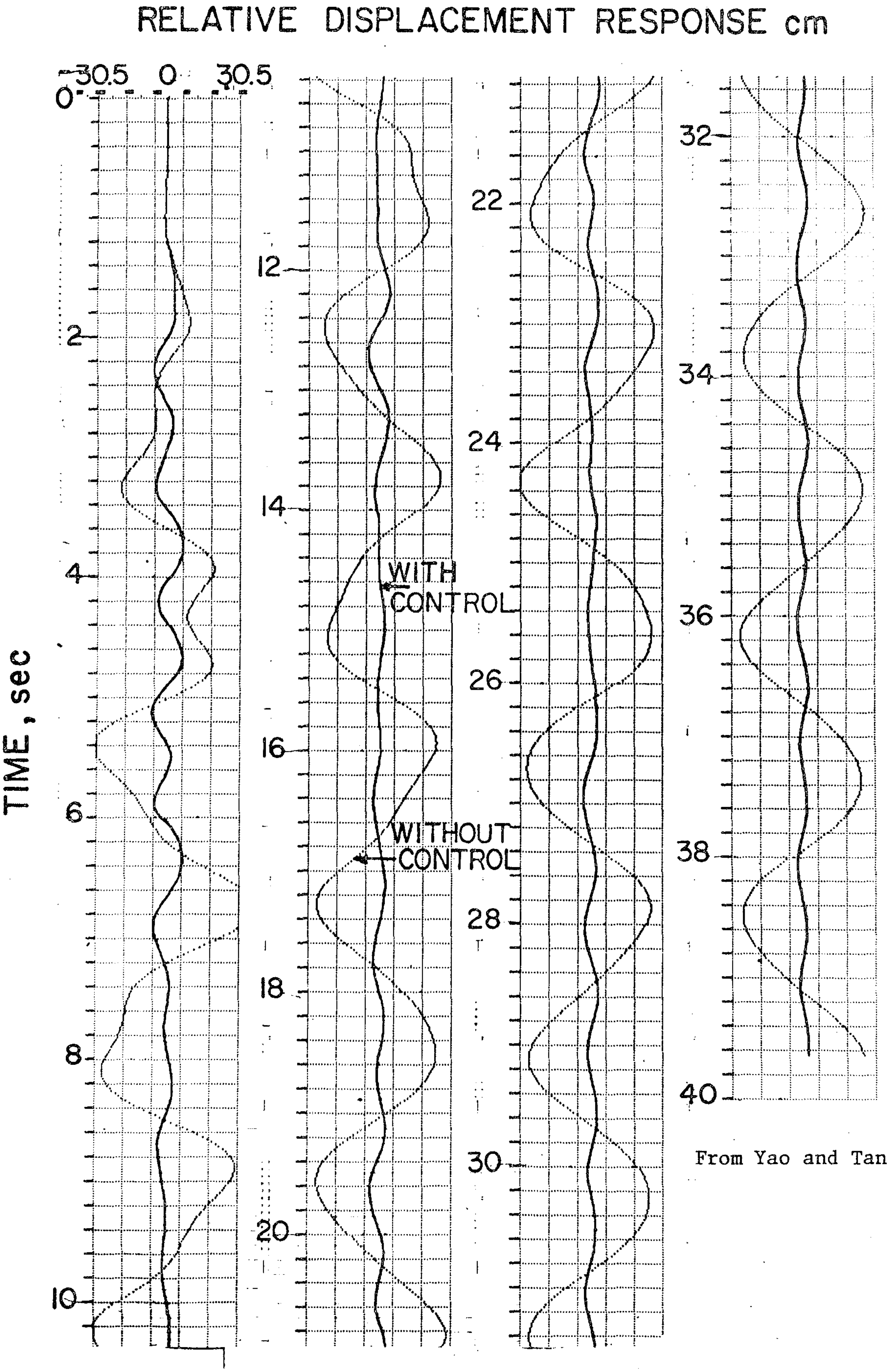


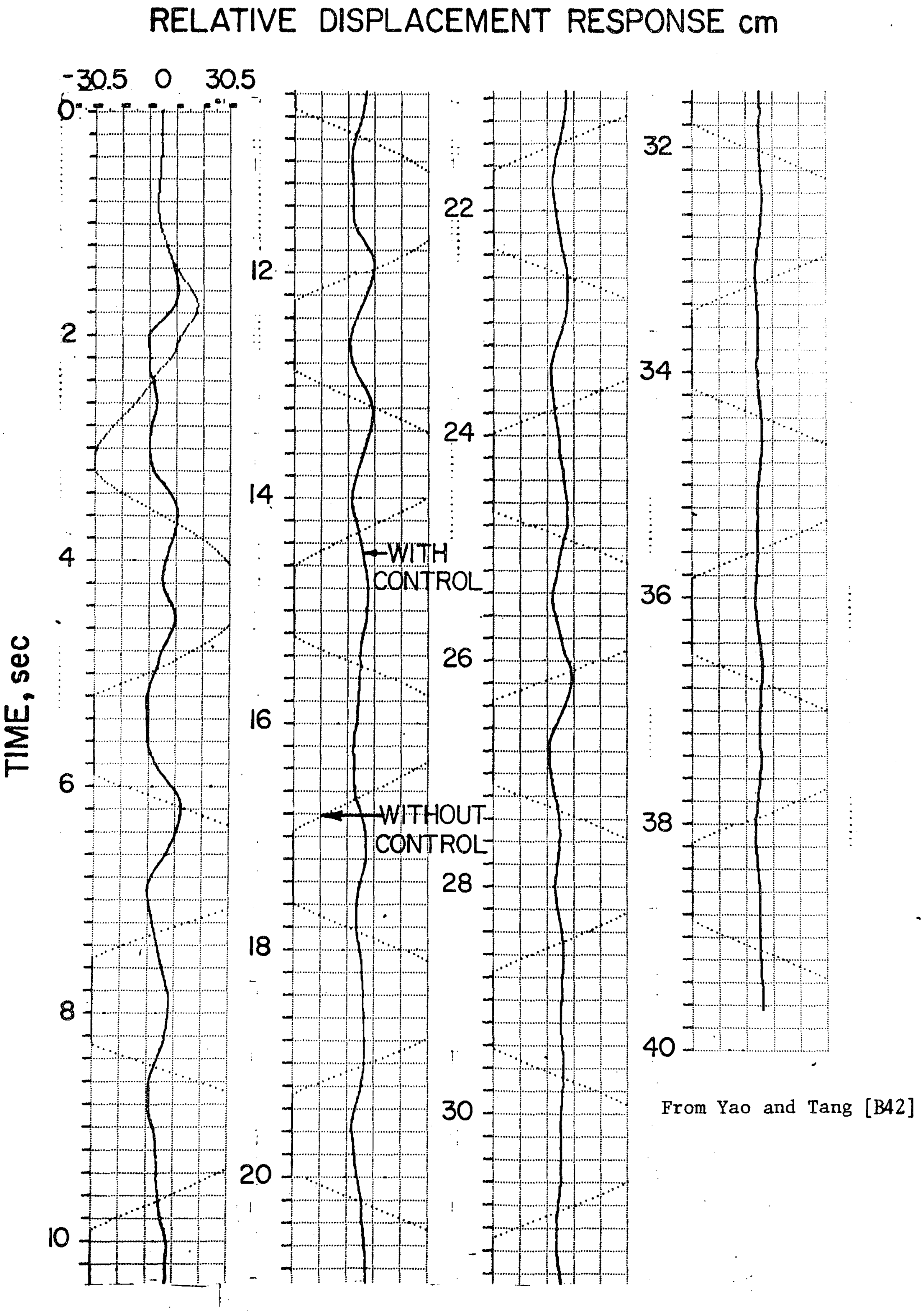
From Yao and Tang [B42]

FIGURE 1(a). DISPLACEMENT OF TWO-STORY STRUCTURE— FIRST FLOOR RELATIVE TO GROUND.



From Yao and Tang [B42]

FIGURE 1(b). DISPLACEMENT RESPONSE OF TWO-STORY STRUCTURE—SECOND FLOOR RELATIVE TO FIRST FLOOR.



From Yao and Tang [B42]

FIGURE 1(c). DISPLACEMENT RESPONSE OF TWO-STORY STRUCTURE— SECOND FLOOR RELATIVE TO GROUND.

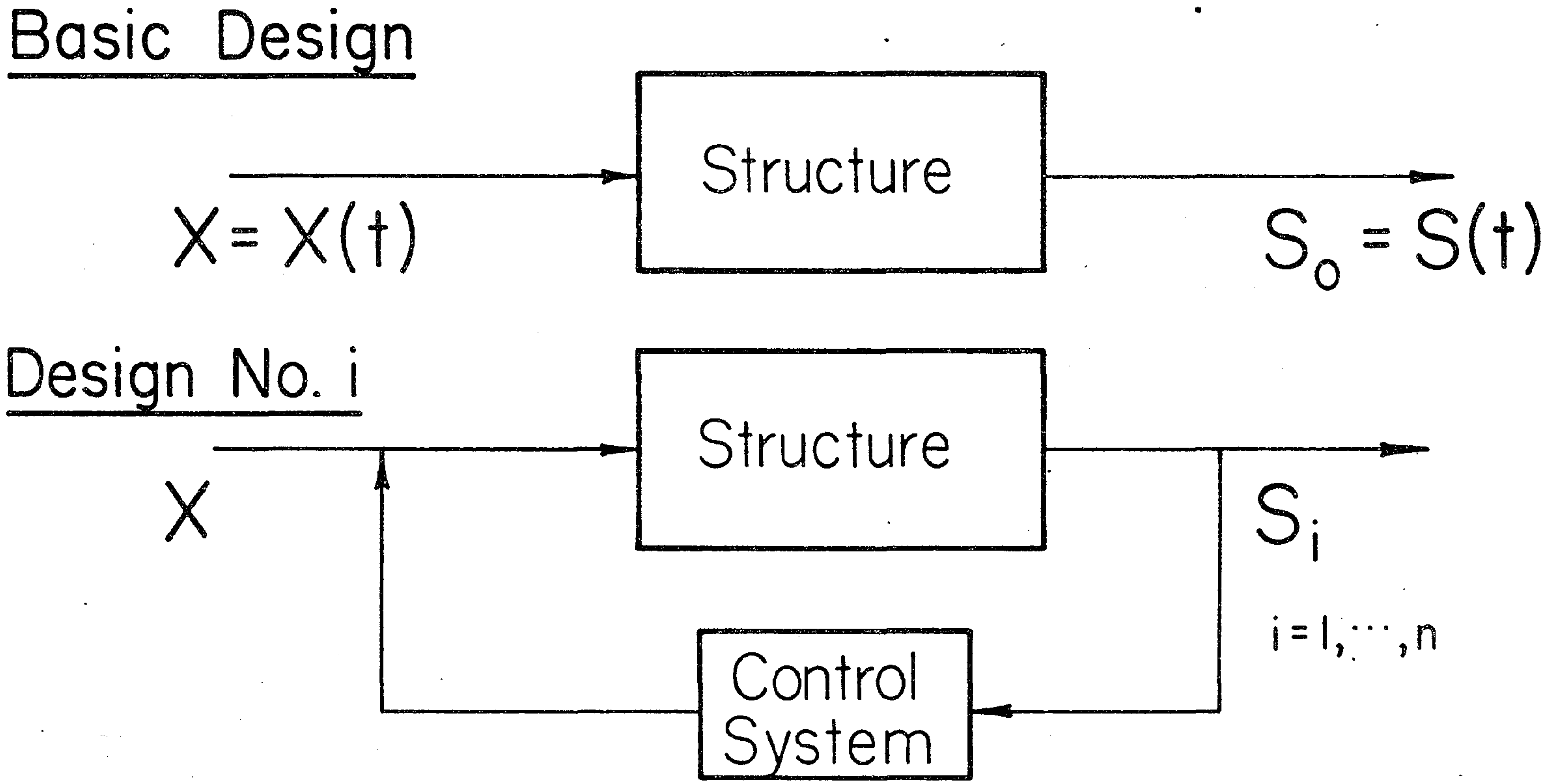
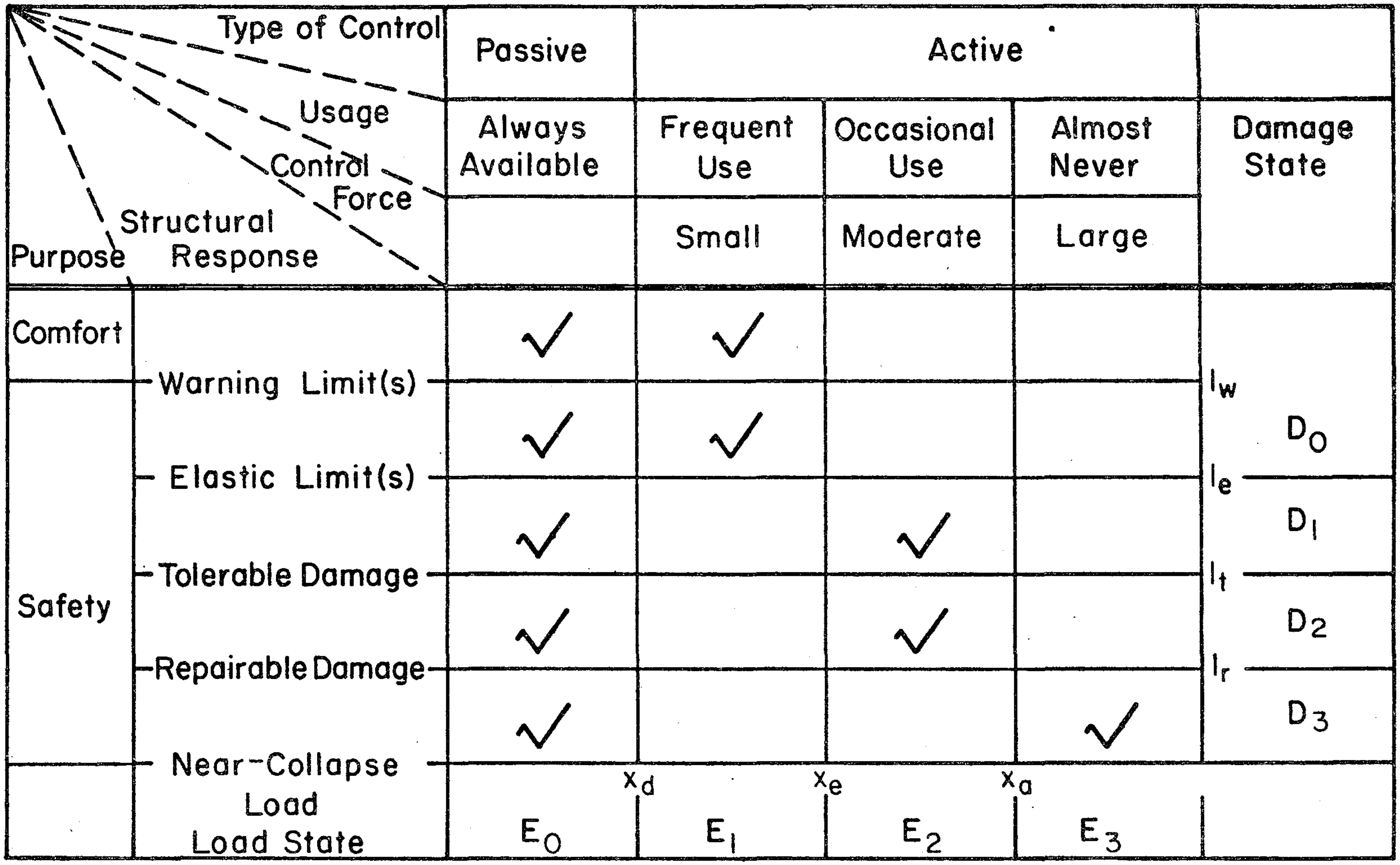


Figure 2. Schematic Diagrams of a Structure With and Without Control System.



Type of Control		Passive	Active			
Usage		Always Available	Frequent Use	Occasional Use	Almost Never	Damage State
Control Force			Small	Moderate	Large	
Purpose	Structural Response					
Comfort		✓	✓			
	Warning Limit(s)					l_w
Safety		✓	✓			D_0
	Elastic Limit(s)					l_e
		✓		✓		D_1
	Tolerable Damage					l_t
		✓		✓		D_2
	Repairable Damage					l_r
		✓			✓	D_3
	Near-Collapse Load		x_d	x_e	x_a	
	Load State	E_0	E_1	E_2	E_3	

Figure 3. Applicability of Structural Control.