SYSTEM IDENTIFICATION, DAMAGE ASSESSMENT AND RELIABILITY EVALUATION OF STRUCTURES

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16. Abstract (Limit: 200 words)
Available literature on the methods of structural identification, damage assessment, and reliability evaluation are reviewed and summarized and the possibility of combining those techniques into a rational procedure for practical implementation is discussed. The available literature has been presented in tabular form. System identification is a process for constructing a mathematical description or model of a physical system when both the input to the system and the corresponding output are known. For most of the current applications, the input is usually a forcing function and the output is the displacement or other motions of the structure subjected to the forces. The mathematical model obtained from the identification process should produce a response that in some sense matches closely the system's output, when it is subjected to the same input. Several recommended procedures for inspection and safety assessment are reviewed and summarized. It is believed that further development and improvement are possible and desirable. The possible incorporation of system identification techniques into damage assessment is also discussed.

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1. INTRODUCTION

1.1 General Remarks

Throughout the history of mankind, man-made structures for various purposes have been built. Most of these structures served their intended functions well and then expired shortly following their respective lifetimes. A few structures stood much longer with proper construction and maintenance procedures. It is thus of great interest to all structural engineers if methodologies can be developed to assess the usable life of existing structures.

In the very beginning, only empirical approaches were available. Various rules of thumb were established from trial-and-correction experiences and intuitions, and passed on from generation to generation. Later, as conceptual models and mathematical tools become available, structural engineers have combined the use of empirical results and mathematical methods to formulate useful engineering analyses and design procedures.

The primary concern of structural engineers is the serviceability of structures and/or their safety. Whenever a need arises and necessary funding is allocated, a structure can be designed and built accordingly. For most structures, the design procedure is an iterative process. A preliminary design of the structure is usually made first and then analyzed mathematically. Results of this analysis are compared with various deisgn criteria (usually in the form of limit states) for serviceability and safety considerations. If and when the design is found to be either overly conservative or unsatisfactory relative to the design criteria, the initial design is modified. The process is then repeated until an acceptable design of the structure is obtained.

Because the real structure and material properties are exceedingly complex, it is necessary for the engineers to propose an abstract structural model and introduce mathematical approximations. Thus the design and analysis are based on simplified and idealized situations in comparison with the field conditions. In some cases, simulated models of the structure are built and tested for the design conditions to verify or supplement analytical results. However, most of these experiments are also performed on reduced-scale models in a simplified test condition.

During the construction stage, the conceptual design of the structure is transformed into reality. However, uncertainties still exist in the quality of constructional materials, continuity (joints and connections), workmanship, fabrication process, and environmental conditions (e.g. effect of temperature variation and humidity in the curing process of concrete). In addition, nonstructural elements such as partition walls tend to alter the structural characteristics appreciably, since they are usually not included in the idealized modeling during the stage of design and analysis.

To obtain improved ("more realistic") mathematical models for a better simulation of the real structure, response records with or without known forcing functions have been collected and analyzed with system identification techniques during the past decade. By necessity, these tests are usually conducted at small response amplitudes so that the serviceability and safety limitations are not violated. Consequently, the resulting modified mathematical models are limited to the linear range of structural behavior. Only in limited cases, weak nonlinearities have been considered. However, natural hazards such as strong earthquakes and severe hurricanes have been known to cause severe damages to existing structures; and the safety of structures under these conditions is of great concern to structural engineers. At present, it is possible to simulate the structural response to such extreme forces with the use of digital or hybrid computers, and thus to evaluate the serviceability and safety conditions of the structures. Nevertheless, there still eixsts the paradox that (a) the applicability of "realistic" models of the structure are limited to small-amplitude response range, (b) the catastrophic loading conditions are likely to cause the structures to behave other than the linear or "near-linear" responses which are usually assumed, and (c) the severe loadings may cause serious damages in the structure and thus change the structural behaviors appreciably. 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 to salvage its residual values.

1.2. Objectives and Scope

The objectives of this report are to (a) review and summarize the available literature on the methods of structural identification, damage assessment and reliability evaluation and (b) discuss the possibility of combining these techniques into a rational procedure for practical implementation.

System identification is a process for constructing a mathematical description or model of a physical system when both the input to the system and the corresponding output are known. For most of the current applications, the input is usually a forcing function and the output is the displacement or other motions of the structure subjected to the forces. The mathematical model obtained from the identification process should produce a response that in some sense matches closely the system's output, when it is subjected to the same input. In general, the system identification technique is composed of three parts:

(a) Determination of the form of the model and the system parameters.

(b) Selection of a criterion function by means of the "goodness of fit" of the model response to the actual response that can be evaluated, when both the model and the actual system are subjected to the same input.

(c) Selection of an algorithm for modification of the system parameters, so that the discrepancies between the model and the actual system can be minimized.

The techniques for modeling and numerical calculations have been developed to a high degree of sophistication in all branches of engineering. Particularly, in the areas of electrical and mechanical control system analyses, the identification techniques have found wide ranges of practical application. However, these techniques cannot be readily applied to structural analysis. Because of the large size and mass of most real structures, many common techniques for generating a convenient force input, and hence a suitable system output, are no longer practical for the identification of structures. Only limited source of input, such as vibrations due to earthquakes, strong wind loads, controlled explosions, are possible to generate sufficiently large excitation. Even for laboratory simulations the limitations on the types of structure and the types of response which can be performed in a laboratory are far greater than an electrical system or a mechanical system. In addition, most of the inputs and outputs are random in nature. To extract useful informations from these data posts an entirely new problem to the system identification.

Most existing literatures in the area of structural identification are concentrated in the selection of an appropriate identification algorithm. Numerical examples and discussions are usually based upon simple mathematical models and sometimes idealized input-and-output laboratory data. Some of the techniques are reviewed in the next chapter. Most of these algorithms are adopted from the existing methods used in other branches of engineering.

Although to determine the form of the mathematical model and the key parameters for the identification process is critical to the accuracy and usefulness of the results, considerably less work is available in the area of modeling technique. Some of the existing discussions in structural identification adopted the usual "black box" assumption for a single input and a single output relationship. Within the similar mathematical framework, lumped-mass systems have often been assumed to represent the real structural behavior. For continuous models, most of the models were also limited to one-dimensional behavior. For example, tall buildings were assumed as cantilevered beams with one lateral displacement and bridges as simply-supported beams. Validity of these simplified models and their ranges of practical application are questions which remain to be answered. More importantly, because of the obvious deficiencies in the present modeling techniques, it is strongly doubtful that many of the high-power algorithms for identification calculations have any practical significance. In Chapter 2, some of the modeling techniques are reviewed and summarized.

In Chapters 3 and 4, several concepts of damage assessment and reliability evaluation for structures are discussed. One of the main difficulties in the assessment of structural damages is that the definition of damage is still ambiguous. Various types of external or internal sources may cause damage in a structure. The influences of such damage to the safety and serviceability of the entire structure are obviously different. For example, cracked plaster, shattered partitions, and even cracks in a floor beam in a building after a strong earthquake may only call for a mere facial repair without significantly changing the life and safety of the building. However, a weakened base column may require a major reconstruction project. Thus, it is highly desirable that a uniform and consistent method of assessment can be defined to evaluate the damage existing in any structure. A time-dependent, damage-combined, index function is proposed in this report. The function is related to the damage in each structural element. A successful damage index can be used (a) to assess the extent of damage in a structure caused by a strong earthquake and (b) to evaluate the future damageability of the structure. The damage index may thus be used in the decision-making of the types of maintenance and repair work needed for a given structure.

Furthermore, if the damage function becomes available, a reliability analysis may be conducted to answer some important questions such as "How reliable would a structure be during an earthquake?", or "How reliable is a building in the next earthquake if some of the damages are left unrepaired?"

2. CLASSICAL TECHNIQUES IN STRUCTURAL IDENTIFICATION

2.1 General Remarks

Modern system identification techniques have been widely used in all branches of science and engineering to identify characteristics of a physical system. However, their applications in civil engineering structures have drawn much attention only in the last two decades. It is in part due to the recent needs in the structural design and in the safety assessment of long bridges, high-rise buildings, and some critical components in nuclear power reactors. It is also due to the analytical difficulties related to the complex nature of the problem. Some of the techniques needed to analyze the identification problem have only become available to structural engineers in recent years.

As in other fields of engineering, the primary concern in the structural identification should lie in the proper choice of a mathematical model which can best represent the characteristics of the structure. In most of the existing literatures, a set of differential equations (lumped-mass model and simple continuous model) or a transfer function (black box model and lumped-mass model in frequency domain) are proposed to formulate the structure behavior. It contains a set of parameters to be identified from the response data of the real structure excited by a known disturbance. A schematic diagram is shown in Fig. 1.

Depending on the type of structural response and the type of disturbance, the differential equation may be assumed to be linear or nonlinear. Some of the parameters may also be disturbance-dependent.

Table 1 briefly summarizes the forms of mathematical modeling assumed in the articles surveyed in the subsequent sections [15]. The methods of analysis used in the articles for parameter identification are also listed. In general, depending on the types of disturbance and structural response, the structural parameters are defined either in the real time domain or in the frequency domain.

In Sections 2.2 through 2.4, the common forms of linear and nonlinear models in structural identification are reviewed. In Section 2.5, brief summaries of the applications of these models in some articles are given. A table is given to categorize various identification problems discussed in the existing literature with regard to their excitations, mathematical models and structural responses. Tables of this form may be used to assist engineers to search for a simple, suitable, and accurate approach for their specific structural problems.

2.2 Linear Models

Because of their simplicity, the linear lumped-parameter models are the most widely used models in structural identification. More complex models such as the linear continuous-parameter models and nonlinear-parameter models are generally used only when the lumped-parameter model has failed to provide an adequate representation of the structural behavior. However, the simple lumped-parameter models are not without restrictions. For lumped systems or continuous systems with lumping approximations the applied disturbance must also be discrete. This is in contrast to the disturbances allowable in a continuous system; they can be either discrete or continuously distributed.

The lumped-parameter system may be mathematically interpreted as the finite-difference discretization or approximation of a continuous system. Thus it avoids a major difficulty in using experimental data to define a continuous functional parameter.

It should be noted that the parameters in a lumped system need not have physical meansings. Thus, the parameters commonly used in a mathematical model, such as stiffness, mass, and damping do not necessarily represent the material properties and mass distribution of a real structure. In a model representation, the geometry, material properties, interactions between various structural elements, boundary conditions, etc. are all "lumped" into the parameters assumed. Thus, the parameters are combined empirical indices, which are valid only for the particular excitation and structural response used in the identification process. To extend the model to include some physical inputs, continuous models have been assumed to give a more rational approximation of the real structural behavior. These models are usually formulated in the form of differential or integral equation. For numerical calculations, the equation is usually discretized by using the finite-difference techniques. Then the system is again reduced to a discrete-parameter system. In the subsequent numerical calculation, such a system is usually more difficult to handle as compared to that involved in a direct lumped-mass model. An alternative form of discretization involves the use of finite-element method to represent a real structure. The versatility of the finite-element method may prove to be most advantageous when two- or three-dimensional structural problems are considered. Its application in the structural identification has only been explored very recently.

Referring to Table 1, the majority of existing work employs the direct lumped-mass model. The lumped-parameter analysis has the advantages of simplicity and easy accountability of the system's nonuniform properties. However, for complicated physical systems, in order to obtain a good "fit", the model may require a large number of parameters. The larger computer capacity and lengthy computer time required may restrict the applicability of the models.

In the following, the common methods of analysis used in the articles surveyed in Table 1 are briefly summarized. These methods can be applied either to the lumped-parameter system or to the continuous system.

a. Modal Expansion:

The structural responses (e.g. displacements) are expressed in terms of the shape functions for the normal modes. The equations of motion describing the structural model are usually decoupled, and the formulation can be written in terms of generalized coordinates. The solutions (i.e. parameter values) are readily available [7,18,47,50,41, 42,49,67,68,79]. It is also possible to extend the method to problems involving non-proportional damping with expansions in terms of non-normal modal shape functions [13,44].

b. Transfer Function:

It is convenient to define the physical characteristics of a structural system in the frequency domain. A transfer function, defined as the ratio of the response function to the excitation function in Laplace domain, is usually taken to represent the structural model for linear and time-invariant system. The physical interpretation of the inversion of a transfer function may be taken as the response of a structure due to a unit impulse. The transfer function is usually rewritten in an algebraic form with coefficients to represent the combined effects of spring constants, masses, viscous coefficients of a linear spring-mass structural behavior model. Since the functional form and the coefficients have no direct physical correspondence, it is generally called a "black box" approach.

The definition of a transfer function is not limited to the Laplace domain. For the convenience of computation, Fourier transformations have often been employed to identify the structural model parameters in the frequency domain. For example, the frequency response of structural model can be obtained directly from the power spectral density functions of the excitation and the structural response, if the transfer function is written in terms of the Fourier transformation parameter. Estimation beased on the finite Fourier transformation [81] has the advantage of minimizing truncation errors. Fast Fourier transformation [41,75] provides an appreciable reduction of computation time and reduces the round-off errors.

c. Estimation Methods:

Various least-squares estimation methods (including repeated and generalized least squares), the instrumental variables method, the maximum likelihood estimation, and the tally principle have been used to handle linear models in structural identification. The least-squares estimation minimizes the summation of square errors between the predicted response and the measured structural response. In the generalized least-squares method, the criterion function for evaluating the "goodness of fit" is the summation of square generalized errors which is defined to include the additive noise covariance matrix. Repeated least squares method modifies the usual least squares procedure by increasing the order of the mathematical model in an iterative process until the accuracy is achieved. Though the validity of these methods has not been proved formally, they have been applied to structural identifcation problems with satisfactory results.

The instrumental variables method applies to the problem of bias with noise-polluted responses [77]. The method involves an iterative process in the calculation of revised estimate and instrumental variables matrix function. The maximum likelihood method is widely used for estimation in statistics. It determines the parameter estimate by minimizing criterion function through an iterative procedure. The method appears to have the advantage of providing the best estimation for a wide range of contamination intensity in the external excitation and the structural response [35,60,77,81].

The estimation methods are generally applied to the time-domain analyses. It usually involves complicated iterative procedures. However, these methods are not limited to linear models only. They can be used to treat nonlinear models for which the modal expansion and transfer functions in frequency domain are no longer defined.

2.3 Nonlinear Models

In contrast with the linear models, very little seems to have been developed in the nonlinear domain. It is in part due to the mathematical difficultues involved in handling the nonlinear terms. Some of the common techniques in dealing with linear systems, such as the modal expansion and transfer function, do not seem to be appropriate in the nonlinear case, though it is well-known that the modal expansion analysis can be applied to weak nonlinear problems to obtain approximate solutions. It is also because the current developments in structural identification have mostly dealt with structural parameters with limited range of application or parameters for highly simplied structural behaviors. For example, in the evaluation of vibratory parameters of structures, the models are often limited to small-amplitude response range and time-invariant structural behaviors. However, as discussed in Chapter 1, the catastrophic loading conditions such as strong earthquakes and windstorms are likely to cause the structure to behave beyond the linear range of responses which are usually assumed. More importantly, the severe loadings may cause serious damages in the structure and thus change the structural behaviors appreciably. Therefore, it is not difficult to envision that the nonlinear model may play a much more important role in the future development of structural identification.

The nonlinear model can be either linear-in-the-parameters or nonlinear-in-the-parameters. If filtering method is employed, only an a priori modeling assumption needs to be made. The remaining modeling problem in choosing an excact mathematical description for the nonlinear function can be determined by a minimizing approach. A number of methods of analysis are available in the nonlinear optimum system control theory. Although it is unproven, some standard techniques have been recommended for use in structural identification as follows:

a. Invariant imbedding and dynamic programming filters [22,32]:

Using the theory of invariant imbedding, a best a priori estimate can be obtained by minimizing an error function. The method is applicable to some general boundary conditions. Dynamic programming filter is a more general method with the invariant imbedding as a special case. Instead of going through the Euler-Lagrange equations to determine the best estimate that minimizes the error function, dynamic programming may proceed directly. Application of dynamic programming uses the decomposition of the error function and leads to a system of partial differential equations.

b. Least Squares Filter [23,24,25]:

The optimal least squares filter satisfies the governing differential equation which describes the structural model and minimizes the quadratic error function. The error function is defined in terms of observed error vectors (weighting matrices) and the best *a priori* estimate of the parameters.

c. Gauss-Newton Method [23,24,78]:

The method belongs to the general family of quasilinear method based on a linear expansion of the system variable around an available estimate of the variable. If the calculation is convergent, it converges quadratically. However, the convergence is not guaranteed.

d. Direct Method [23,24]:

If an accurate acceleration measurement is available, a direct approach may be used without the need of an initial estimate of the coefficients. For some cases, it requires only partial estimates. The parameters are determined by directly minimizing the quadratic error function. The method appears to be efficient in computation, particularly for nonlinear models with a single degree of freedom.

e. Extended Kalman Filter [60,71]:

The Kalman filter has been used to obtain optimum sequential linear estimation and an extended filter deals with nonlinear filtering. Its good approximation for high sampling rates has been demonstrated in simulation studies of parameter estimation.

f. Maximum Likelihood Method [60]:

The method has been applied to both linear and nonlinear systems. It can handle both the measurement noise and the process noise, and may also be used to estimate the covariances of the noises. In [60], it has also been suggested that the extended Kalman filter may be introduced in the calculation of the likelihood function.

g. Wiener Filter [57]:

An input-output relationship of multiple integral form is assumed to represent the model. The kernel functions which represent model parameters can be estimated by a cross-correlation technique. In theory, the relationship can be written in Laplace domain and thus the kernels are identified in terms of the Laplace parameter. Their values in real time domain are then obtained by the usual inversion techniques.

2.4 Identification Parameters in Structural Dynamics

Most of the literature which has been surveyed in this report deals with the linear lumped-parameter model or the linear continuous model. The formulation is given in the form of a set of linear equations of motion:

$$m\ddot{x} + c\dot{x} + kx = F \qquad (2.1)$$

Where x is the structural displacement response matrix, F is the excitation matrix (usually the external forces), m is the mass matrix, c is the damping matrix and k is the stiffness matrix. Hence, the parameters to be identified are usually the m, c and k matrices. As discussed previously, these matrices do not need to be physically related to the real mass distribution and material stiffness.

The form of nonlinear models generally varies with the type of excitation and the algorithm employed for numerical calculation. One of the direct extensions of the linear model can be obtained by assuming

$$m\ddot{x} + h(\dot{x},x) = F \qquad (2.2)$$

Where nonlinear function h may be taken as an odd algebraic function in $\dot{x}$ and x [24,25], i.e.

$$h = a_1 x + a_2 x^3 + a_3 \dot{x} + a_4 \dot{x}^3 \quad (2.3)$$

Integral form of the formulation of the excitation-response relationship has also been used. It is convenient when transfer function is being used to handle the linear model. In an integral formulation, instead of using three constant-parameter matrices, i.e., m, c and k, the model characteristics are lumped in a kernel function $h_1(\tau)$ in the following form:

$$x(t) = \int_0^\infty h_1(\tau)\ F\ (t - \tau)\ d\tau \quad (2.4)$$

It is easy to extend the integral formulation to include the nonlinear kernels. For example, a second-order model has the form [57],

$$x(t) = \int_0^\infty h_1(\tau)\ F\ (t - \tau) d\tau + \int_0^\infty \int_0^\infty h_2(\tau_1, \tau_2) x(t - \tau_1) \cdot x(t - \tau_2) d\tau_1 d\tau_2 \quad (2.5)$$

Table 2 [15] categorizes the types of forcing function, structural model, and structural response considered in the articles surveyed in this report. Briefly, they can be summarized as follows:

a. Excitation:

The excitation function for structural identification can be deterministic or random. As a practical example, the ground motion recorded during a specific earthquake is deterministic. The data may be periodic or non-periodic, analytically formulated or represented in digital form. However, future ground motions possess the unpredictability and variability inherent in a random process. Some common methods to generate random inputs make use of statistical theories such as the maximum likelihood estimate [35], statistical moments [18,50], and the mean squares spectral density [35,75,83].

A deterministic input can either be an ambient force or an imposed force. Some of the ambient forces, such as ambient winds are usually low-level excitations and distributed over the entire structure. The resulting structural responses are difficult to analyze. As an aid to the convenience and accuracy of structural identification, controlled excitations may be imposed on a structure. These can be in the form of transient impulse or steady-state sinusoidal types of vibration [72]. In addition, the inputs may be mathematically written in the form of spectrum function or functions continuous in time. These two forms can also be converted into each other by using the usual Laplace or Fourier transform.

The environmental noises can cause significant errors in the handling of measured data. For structural identification, the higher the noise/input ratio, the greater the error in the calculation of the damping matrix [47]. The process noise and the observation noise can be treated by using standard statistical means such as the maximum likelihood method [60]. The noise can be treated as sinusoidal [72], random, or ambient. It is often represented as a white noise which has a constant spectral density independent of frequency over the range considered [14,32,35,47,50,57,60,81,84]. Some investigators oppose the use of white noise because such a process is not physically realizable. For the conveneinece of analysis, the noise is often considered as stationary, though it may be non-stationary in the measured data.

b. Models and Parameters:

Most of the literature deals with analytical methods for the determination of the elements of the mass, damping,and stiffness matrices in the linear equations of motion for lumped-parameter models. The mass matrix is in general the most important parameter in structural dynamics since it affects the inertia term. Elements of the mass matrix are usually assumed or partially determined in most of the identifying process. Modifications of the mass matrix can also be incorporated into the algorithm to yield a better fit of the data. When rotational motions are included, the correspondig mass matrix includes terms with the dimension of mass moment of inertia.

The damping matrix represents the combined effects of material dissipation, interlay slip, boundary damping, and other factors which affect the attenuation and duration of vibratory motion. When the modal expansion technique is used to decouple the equations of motion, the damping matrix is often taken to be proportional to the mass matrix, to the stiffness matrix, or to a linear combination of both, for the convenience of identification. The assumption of proportional damping reduces the number of elements to be identified and thus greatly simplifies the numerical calculations involved [7,41,68]. Without recourse to the limitation of proportional damping, identification algorithms using non-normal modes [4,44] and estimation methods [4,13] have also been suggested.

Almost in all articles surveyed in this report, the parameter matrices are treated as being time-invariant. To consider structural damages caused by strong earthqukaes or to examine the structural deterioration with time, stiffness and damping matrices should be identified as time-dependent functions. No such example has been found in the available literature to-date.

c. Response:

Structural responses are usually recorded in acceleration, and/or displacement at a particular location or locations. However, not all of the information is available or sufficiently accurate for identification purposes. For example, the structrual response due to earthquake is often given in acceleration. If the record is relatively noisefree, velocity and displacement data can be obtained by a direct integration. In general, the data are quite accurate. However, data obtained through differentiation of other records are not usable for obvious reasons.

2.5 Survey of the Models and Methods

Brief descriptions of the articles surveyed in this report are given in Table 3. Information concerning the excitation, structure, and response of the experimental example and those of the mathematical model are listed for the purpose of comparison. In the following, some of the selected articles are summarized to provide more detailed information about the state-of-the-art in structural identification.

A linear, discrete structural model was studied by using the modal expansion method in the article by Berman and Flannelly [7] for structures having a relatively large number of points of interest and a frequency range of interest influenced by a relatively small number of normal modes. The article pointed out a basic and inherent difficulty in attempting to use test data to define a finite degree-of-freedom model of a continuous system. In the numerical process of structural identification, the governing equations are ill-conditioned and thus the solutions are very sensitive to small measurement errors. To avoid such difficulties, a structural model was introduced which contains fewer degrees of freedom (normal modes) than coordinates (points of interest). The parameters of this "incomplete model" are obtainable from the limited, but quantitative, test data. The physical example selected for testing the theory was a simple, thin overhung beam in transverse vibration. The beam assumed a constant bending rigidity and a constant damping ratio and had 18 lumped masses to simulate a uniform mass distribution. The number of points of interest were taken to be 11 and the number of normal modes was 3. The sensitivity of the method of identification was tested by the effects of parameter changes due to two assumed conditions: (a) assuming that a large mass was added at the tip, and (b) assuming that a spring was attached to the ground at the tip. The results appeared to be satisfactory even when the data were polluted with simulated test errors.

In reference [13], Caravani and Thomson proposed a numerical technique which identifies in an optimal sense the damping coefficients of a linear lumped-parameter system whose frequency response is known over some frequency range. The identification is perfomred in iterative manner by processing one frequency point at a time. Again, the standard modal expansion techniques were used and the analysis was performed in frequency domain. The general procedure is quite similar to that introduced by Hall, Calkin and Scholar [37]. Example problems were given for a two-degree-of-freedom system with non-proportional damping matrix, and for a lumped system with six degrees of freedom with viscous damping.

A recursive least squares time domain approach for structural identification was suggested by Caravani, Wasson and Thomson [14]. The method preassumed a structural model and the parameters of the model. The parameters were identified from a series of time observations of the structural response taken at various points of the structure. The data were the structural acceleration at the recording point resulting from an excitation. The least squares estimates at each time increment by a recursive formula based on the minimization of the least squares error function.

A method for the statistical identification of a structure was formulated by Collins et al. [18]. It used measurements of natural frequencies and mode shapes to modify the structural parameters of a finite element model. The method assumed values of the structural properties as the starting point. Then, these values are modified to make the modal characteristics confirming to those observed in test. Accuracy of the values of the test data and the engineer's confidence in the values of the model properties are incorporated in the procedure by a statistical approach.

Two numerical examples were included in [18] to demonstrate the advantages of the statistical approach. The first example contained a free-free beam which was modeled by using two finite elements and six generalized coordinates. The system contains two rigid body modes and four elastic vibration modes. It is a linear model and the standard modal expansion method was employed. And thus structural parameters were identified for the normal modes. The second example was chosen to be a lateral vibration model of Saturn V rocket. The rocket model was represented by 28 beam elements having both bending and shear stiffnesses. The random bending and random shear stiffnesses in each element summed to a total of 56 structural parameters. Test data were obtained for the first three elastic modes. In this example, the measured frequencies were less than 5% different from the predicted ones to start with and the identification procedure produced convergence to less than 0.03% in three iterations.

In reference [19], Collins, Young and Kiefling surveyed the system identification techniques in the shock and vibration area. A technology tree was developed along two principal branches--the frequency domain and the time domain for the purpose of assisting engineers in matching a particular need with available technology. Specific examples of accomplished activity for each identification category were discussed. Special emphasis was focused on the use of statistical approach in structural identification. Numerical examples for the estimation of the stiffness of a spring-mass chain and a two-degree-of-freedom system were given by using the weighted least squares method.

A statistical approach to estimate parameters of a linear structural model having certain modes and frequencies which are as close as possible in a weighted least squares sense to the corresponding experimental data was also adopted by Hall, Calkin ans Cholar [37]. In their lumped linear model, the mass matrix was assumed to be known and an optimization procedure including the minimization of a quadratic cost function was adopted to estimate the elements of the stiffness matrix. A non-uniform beam with cylindrical cross section was taken as the example to test the iterative procedure. In general, the numerical procedure was quite similar to that used in Ref. [13].

The efficiency of constructing appropriate nonlinear model to identify the structural behavior of a three-story steel frame tested on a seismic table was discussed by DiStefano and Pena-Pardo [22]. An invariant imbedding filter was utilized to achieve an optimum estimation. The paper first introduced a viscous damped linear model. The model parameters were identified by using one set of test data and then the identified parameters were used to predict responses due to two other excitations. The results were found to be unsatisfactory. A nonlinear stiffness model was then fitted to the same test data and a considerable improvement in overall predictive quality was obtained. The nonlinear system assumed for numerical example included three initial displacements, three initial velocities, and four unknown parameters. A given error functional representing the deviation of predicted values from the observations was minimized. An iterative procedure yielded convergent values of the unknown parameters.

In a series of articles, DiStefano and Rath [23,24,25] discussed various filtering techniques and the nonlinear modeling applied to problems in structural seismic dynamics.

A least squares procedures for the identification of a nonlinear single-degree-of-freedom system was presented in Ref. [23]. The procedure does not require a prior estimation of the structural parameters. The nonlinear equation of motion contains cubic terms in displacement and velocity, and thus requires four structural parameters. Several numerical cases were studied in the article. The North-South component of the 1934 El Centro earthquake was used as the excitation input. Three different types of observations were considered: (a) the observations are available for the displacements, velocities and accelerations at some points of the structure, (b) the observations are available for the displacements and time histories of the response, and (c) only the acceleration records are measured. It can be seen that the third case is probably the most common record available for earthquake engineering application. Unfortunately, this was also the least accurate case for the algorithm proposed in the article.

Three different methods were presented in Ref. [24] for the similar types of nonlinear equation of motion. The first one was a direct approach. The identification of the structural parameters was accomplished by minimizing directly the quadratic form of least squares function. The method is simple and does not require an initial estimate of the parameters, but it requires the measurements to be very accurate. The other two approaches are based on methods of control and optimization theory. A filtering method requires that the solutions satisfy a differential constraint, and that the least squares function is a minimum. The Gauss-Newton approach uses a modified least squares function. Both methods require an initial estimate of the parameters and an iterative procedure to achieve convergent results. The seismic record used in Ref. [23] was also employed in [24] as the input data for numerical calculation. Although the methods were formulated for systems of multidegree of freedom, only numerical results for a one-degree-of-freedom system simulating the lateral displacement of a shear frame were reported.

The filtering method of structural identification outlined in [24] was also applied for the estimation of parameters associated with two other models in a subsequent developement [25]. One of the models exhibited a bilinear hysteretic loop of kinematic type, and the other a viscous model formulated in the form of a nonlinear differential equation. For the bilinear model, emphasis was placed on the development of an algorithm to bypass the difficulty originated from the indetermination of the identification problem associated with the piecewise linear model employed. The viscous model simulates a steel frame tested on a seismic table. Measured data were used in the numerical calculation. Again, only the systems with one degree of freedom were studied.

Fry and Sage [32] demonstrated the application of the maximum a-posteriori filter to the problems involving continuous-time system identification. Although the article was attempting to identify aircraft stability and control derivatives from flight test data, the identification algorithms appeared to be useful for parameter identification with nonlinear structural models.

The basic algorithm contains the calculation of the maximum a-posteriori estimate of the system state by minimizing a cost function. In the process, it is assumed that the prior statistics (mean and co-variance of plant noise vector, measurement noise vector, and the initial state) are known. If some of these quantities are unknown, they may be replaced by weighting matrices; the resulting estimate is a least squares estimate. The Pontryagin maximum principle was used in the article to solve the optimization problem. A minimized Hamiltonian was imposed as a necessary condition for a minimum cost function.

In Ref. [35], Gersh, Nielsen and Akaike proposed a new, statistically efficient and computationally efficient maximum likelihood computation procedure for determining the period and damping coefficients of linear structural models. The recorded structural response due to random winds or earthquake excitation may be used for the calculation. In the procedure, random data were sampled at regular intervals for digital computation. A parametric discrete time series model was fitted to the correlation function computed from the sampled data by a maximum likelihood procedure. The structural parameter estimates were computed from the time series model in a manner that preserved the statistical efficiency of the estimates. Numerical results were computed from a wind building response data of a nine-story steel frame building.

The Eigenvalue uncertainty of structural parameters was considered by Hart [40] in determining the mean of natural frequency, its standard deviation, and their ratio. The structural parameters were treated as random variables. The numerical procedure used in [40] was an approximation of the one adopted by Collins, Hart, Hasselman and Kennedy [18]. In general, the parameters were estimated first. Then, by minimizing the standard deviation, the estimated values for mass and stiffness of the structural model were obtained. Numerical examples included the analysis of a two-bar truss.

Values of modal damping were analyzed by Hart [41] by using the dynamic records obtained in 12 southern California high-rise buildings during the February 9, 1971 San Fernando earhtquake. Fourier spectrum techniques were used to obtain the damping values in the building normal modes of vibration. Empirical equations, which may be used for siesmic design of buildings, were derived relating the modal damping in steel and concrete buildings to site 0% damped pseudovelocity response spectrum amplitude at each natural frequency of the building vibration. A design procedure was also suggested. It was further suggested that the data may also be applied to nonlinear building analyses.

In Ref. [42], Hart discussed the application of structural identification concept to study the character of the wind loading. Traditionally, the study of wind effect starts by selecting a description of wind forces acting on a building and an analytical model of the building to estimate response. Hart suggested that the study may be started by considering the response and model to estimate wind loading, i.e., an inverse approach. Several numerical algorithms based on both the frequency domain analysis and the time domain analysis were formulated.

Hart and Yao [43] presented a state-of-the-art review of the identification theories and applications in structural dynamics. The authors followed the technical tree developed in Ref. [19] and updated this tree to include publications up to 1976. They also recognized and reviewed some published research work along more philosophical branches. They included identification problems which require a prior structural model with or without a quantification of experimental and modeling errors. The review also contained a brief description of the algorithms and sample data. The article listed 63 references; most of the referenced articles were published in the last decade.

In a short article [44], Hasselman discussed a method for measuring the off-diagonal terms of damping matrix. Such cases may occur when the normal mode method is employed for dynamic analysis of structure, where non-proportional damping is assumed and thus the damping matrix in general can not be diagonized. The authors suggested a perturbation technique in complex domain. However, no numerical example was given to substantiate the procedure, and the question of measurement error sensitivity was not discussed.

Ibanez, Vasudevan and Smith [46] discussed some new concepts in instrumentation, test procedures, and data processing for structural identfication using vibration testing data. A pseudo-inverse method was suggested to determine the optimum placement of vibrators and accelerometers for identifying Eigen parameters of the structure, as well as the unknown forces acting on the structure. Several case studies were given including the vibration records of the San Diego Gas and Electric Company office complex, the United Casualty and Mode Shape Building, and the Bechtel Corporation Office Building.

Transient testing techniques were used in the determination of the frequency response of a structure by Kandianis [47]. The article focused on the effects of extraneous noise on the frequency estimate. The author demonstrated that, when the noise is present either as an additional structural excitation or as structural response, the determinstic approach of transform function representation yields very poor results. It was further demonstrated that, by considering the spectral density function and the autocorrelation function of the response, the noise does not affect the measurement of the natural frequency and damping if the noise is presented in the structural excitation. A new analysis technique was then suggested by taking the unilateral Fourier transform of the autocorreslation function of response. Several advantages of the technique applied to the analysis of transient response were also discussed. The method of analysis is analogous to the technique adopted by Schiff [72] in his analysis of data from ambient and low level excitations.

The dissertation by Klosterman [49] has included a rather complete review of the application of modal techniques. The specific objective of the dissertation was to develop new techniques which relieve some restrictive assumptions concerning the form of the damping matrix and the spacing of the natural frequencies. Based on the normal mode analysis, the algorithm was applied to study systems of which the viscous damping matrix is not proportional to the mass and/or stiffness matrix. The case involving a hysteretic damping matrix was also discussed.

The moment technique for parameter identification, suggested by Kozin and Kozin [50], was based upon the properties of statistical expectations and time averages. It can be applied to nonlinear as well as linear constant systems subjected to random or sinusoidal excitations. To implement the technique, it is necessary to know the excitation and the complete vector associated with the system. The authors considered an illustrative example by applying the method to a five-degree-of-freedom linear spring-mass system. The complete and noise-free state vector of the model was obtained from a digital simulation of the actual model for a given excitation and the parameters subsequently estimated. The estimated parameters were found to agree well with the actual parameters. However, the article did not include the measurement error which may affect the results.

In a series of two articles, Marmarelis and Udwadia [57,84] studied the Wiener technique of nonparametric identification. The structural system was represented by an input-response relationship of integral form. For nonlinear system a double integral term was added and thus the system is characteriazed by two kernel functions. For a given input, the kernels were estimated to obtain simulated responses. By comparing with the measured responses, the mean-square-error reduction by using the cross-correlation technique and system feedback yields a refined estimate. The algorithm was applied to the identification of a reinforced concrete building with input-response data obtained during a strong ground shaking.

In the two articles [58,59], McNiven and Matzen described their application of system identification method to formulate a nonlinear model for representing the seismic behavior of a single story steel structure. The development involved the use of a second order nonlinear differential equation with linear viscous damping, the Ramberg-Osgood type hysteresis, and a modified Gauss-Newton method to minimize an integral squared error function. Shaking table experiments in which a single story steel frame was subjected to several earthquake excitations were conducted to give the necessary numerical input. The results showed that the correlation of the computed accelerations with the measured was excellent.

The paper presented by Mehra, Stepner and Tyler [60] suggested the application of the maximum likelihood criterion as a method of system identification for flight test data analysis. Although the paper is not directly related to structural analysis, the method appears to be applicable to the identification of nonlinear structural models.

A generalized maximum likelihood method which includes the output error method and the equation error method as special cases was applied to flight test data. Accurate fits to the time histories were obtained with the presence of lateral gusts during the test flight. The method was also supplied to nonlinear flight dynamic model with process noise. Some improvements in the evaluation were also suggested by using a multistep input.

Pilkey and Kalnewski [64] treated the process of dynamic force identification as a mathematical programming problem. The authors acknowledged that this approach should be a powerful technique to the identification problem, if the system models, constraints, and objective functions can be expressed as linear functions of the dynamic force. Examples were given to (a) the identification of some earthquake acceleration records from a shock spectrum, (b) the computation of unbalanced forces of rotating shafts based on displacement observations, and (c) the evaluation of upper and lower bounds of the force for shock loaded system in which only peak responses can be observed.

A time domain analysis based on estimation method was considered by Raggett [66]. The algorithm requires initial estimates of natural frequencies and model damping coefficients. A minimum least square error curve-fitting procedure was applied to filtered response data to seek the best fit root-mean-square responses, periods and damping ratios. Examples included the analysis of ground motion records for a 29-story building.

Low-amplitude nondamaging motions were used by Raggett [67] along with the natural mode shapes, frequencies of the structure, and the energy ratios for the identification of various building elements. The total model damping ratio was taken as the sum of the component energy ratios weighted by the respective ratio of peak component potential energy to total potential energy. Using his method, damping can also be treated as a function of amplitude of motion. The accuracy of the damping ratio were found from the accuracy of periods. Results of examples agreed well with observed values.

In the identification of complex structures using near-resonance testing by Raney [68], the sinusoidal force was used as the input, and the steady state responses for frequencies near major structural resonance were obtained. Using a modal transformation, a set of uncoupled equations corresponding to several modes was obtained. Steady state solution for a sinusoidal input was used to determine the system parameters. The test data of Langley 1/10-scale and 1/40-scale models of the Apollo/Saturn V vehicle was used as numerical input.

In a report for literature review, Rodeman and Yao [69] selected nine representative papers to summarize the modal methods in structural identification. The essence of each algorithm was outlined, and its applications and possible difficulties were discussed. Most of the articles dealt with linear lumped-parameter structural models.

Sage's review article [71] is an excellent reference concerning the classification and methodology of the system identification as applied to structural problems. The article also reviewed some general techniques for identification; techniques based on transfer function identification, learning model identification, and identification based on nonlinear filtering. Examples to illustrate the techniques were also presented.

Schiff [72] reviewed test methods and methods of analysis specifically applied to the identification of large structures using data from ambient and low level excitations. Depending on how the field data is obtained, the author categorized the existing methods into three areas: low level forced vibrations, the response from low level ambient excitations such as wind and microseismic shocks, and large amplitude response data resulting from earthquakes. The article included a rather extensive list of reference in the area of testing methods and measurements.

Several practical applications of the system identification method were shown in References [74,75,79]. Sewall [74] applied the linear lumped parameter models and the transfer function representation to simulate the dynamic behaviors of the Penn Centra's electrically propelled Metroliner high-speed train and the suspension system of a linear induction motor for DOT's 300-mph Tracked Air Cushion Research Vehicle. Shapton et al. [75] applied a similar model to identify the dynamic characteristics of four different types of machine tools. The latter article has also included a long list of references related to the other case studies of the application of identification method to tool technology.

Sparks and Crist [79] applied the linear model and modal method to characterize the response of the Post Office Tower at London, England due to wind loadings.

Shinozuka et al. [77] identified the damping matrix and the stiffness matrix in a two-dimensional model of a suspension bridge subjected to vertical and torsional aerodynamic vibration by applying statistical techniques, such as the least squares method, the instrumental variable method, and the maximum likelihood method, using observed response in the time domain. An autoregressive moving average (ARMA) model was introduced where the observation vector was expressed as the summation of weighted fluctuating components of wind velocity and observation error vectors. The maximum likelihood method provided the best estimation for cases including a wide range of intensity of contamination (e.g. noises) in the input and the output data.

The identification of a random two-compartment model from kinetic data in pharmacokinetics for estimating the properties of the random rate constants was presented by Soong and Dowdee [78]. The method was based upon an estimation algorithm which estimates the statistics of a random exponential model with random amplitudes and time constants.

The article by Sweet, Schiff and Kelley [81] discussed the problem of identifying structural parameters of large structures for which their low level responses require special treatment. In this paper, the authors suggested to average the response of the structure to each of a sequence of impulses, where the repetitive impulsive loading was also suggested as the structural excitation. Finite Fourier transforms were used. The parameters which appeared in the structural model were estimated by the use of maixmum likelihood method. Emphasis was placed on estimating the damping parameter and its associated confidence interval.

Modal frequencies of a car were obtained from the analysis of the acceleration time-histories by Talbot et al. [83]. A vibration power spectrum of a point on the car structure driven on the road was taken. The statistical error analysis and the cross spectral density were used to find the phase which was essential for the identification of mode shapes. Then, frequencies, at which a peak on a coherence vs. frequency plot occurred, were taken. Because the selection of modal frequencies is entirely automatic, this program has been used commercially with success in saving hours of laboratory work.

A continuous model was estimated from available earthquake records by Udwadia and Shah [85]. It was assumed that the mass distribution per unit height of the structure was known and the stiffness distribution per unit height was estimated from measurements at the base point and some other points of the structure. The method involved iteratively changing the stiffness estimates based on an initial estimate and the observed response to a given ground motion. The new estimate was obtained by minimizing the error criterion function.

Experimentally determined unit impulse response functions were used to determine the transient responses of a linear mechanical system subjected to arbitrary excitations by Warkulwiz [87]. This technique can also be used for studying transient responses and arbitrary excitations. The algorithm was claimed to be quick, cheap, and reliable.

3. DAMAGE ASSESSMENT OF EXISTING STRUCTURES

3.1 General Remarks

Traditionally, structural engineers are responsible for the design and analysis of the structures, which are then constructed under the management of general contractors. Following the completion of the construction process, the use and maintenance of most civil engineering structures do not require the service of structural engineers until the occurrence of some disastrous event such as strong-motion earthquakes or severe wind storms.

In Figure 2, a schematic diagram is given to illustrate the beginning portion of the lifetime of a structure [93]. At time t_o, the construction of the structure is completed. Suppose that a strong-motion earthquake occurs and causes some damage at time t_1. Structural engineers may be requested to inspect the structure and may perform nondestructive tests at time t_2. The resulting data can be analyzed for the purpose of making damage assessment. Alternatively, the structure can be inspected and tested without having experienced any disastrous events as a routine and periodic maintenance procedure as a safety precaution. In any event, a decision can be made on the basis of such damageability evaluation or damage assessment as to the type and extent of repair or strengthening required. This cycle can be repeated until the structure is no longer needed or destroyed beyond repair.

The objective of this chapter is to (a) review and summarize several existing methods of damage assessment and damageability evaluation, and (b) discuss the possibility of developing a new methodology incorporating available techniques of system identification as well as the concept of structural reliability.

3.2 Damage Assessment and Damageability Evaluation

An investigation can be initiated by one of more interested parties whenever there are signs of distress or failure in a structure. Alternatively, existing structures can be examined as a routine and periodic procedure. Typically, these investigations consist of both experimental and analytical studies [11,38]. Recommendations for specific repairs can also be included if they are so requested. The experimental studies can be either field surveys or laboratory tests or both. 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. On the other hand, 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.

In studying the building damage resulting from the Caracas Earthquake of 29 July 1967, Seed et al. [73] used several quantities representing building damage for the purpose of comparison. For a given region, the structural damage intensity denotes the ratio of the number of damaged buildings to the total number of buildings in this region. For individual buildings, the ratio of maximum induced dynamic lateral force to static design lateral force is used for brittle structures, and the ratio of spectral velocity to lateral force coefficient is used for ductile structures. More generally, Bresler, Okada, and Zisling [12] proposed the use of capacity ratio, c. The quantity $\ell = 1 - c$ is called the leniency ratio. Either of these two ratios can be specified along with permissible time for hazard abatement of three categories of building structures according to their relative importance [12].

Bertero and Bresler [8] 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 [10] 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 degrading resistance, the ratio of the initial to jth-cycle resistance at the same cyclic peak deformation was also suggested.

Wiggins and Moran [89] proposed an empirical procedure for grading existing building structures in Long Beach, California. A total of up to 180 points is assigned to each structure according to the evaluation of the following five items:

1. Framing system and/or walls (0, 20, 40 points). A well-designed reinforced concrete or steel building less than 3 stories in height is assigned a zero-value. On the other hand, an unreinforced masonry filler and bearing walls with poor quality mortar is assigned a value of 40 points.
2. Diaphram and/or Bracing System (0, 10, 20 points). As an example, zero values corresponds to well anchored reinforced slabs and fills. On the other hand, incomplete or inadequate bracing systems correspond to the high 20 points on the scale.
3. Partitions (0, 10, 20 points). Those partitions with many wood or metal stud bearings rate zero points. On the other hand, unreinforced masonry partitions with poor mortar will draw 20 points.
4. Special Hazards (0, 5, 10, 15, 20, 35, 50 points). The high hazards include the present of non-bearing, unreinforced masonry walls, parapet walls, or appendages.
5. Physical Condition (0, 5, 10, 15, 20, 35, 50 points). The high hazards include serious bowing or leaning, signs of incipient structural failure, serious deterioration of structural materials, and other serious unrepaired earthquake damage.

All of these assigned points are summed for each building thus inspected. Rehabilitation is not required if the sum is less than 50 points (low hazard). Some strengthening is required if the sum is between 51 and 100 points (intermediate hazard). Demolition or major strengthening is necessary when the sum exceeds 100 points (high hazard).

Culver et al. [20] presented the field evaluation method (FEM), in which a rating of 1 to 4 is assigned for each geographic location rating, structural system rating, and nonstructural system rating. Then a composite rating, CR, is computed. The building is said to be in good condition, if CR < 1.0; in fair condition, if 1.0 ≤ CR ≤ 1.4; in poor condition, if 1.4 ≤ CR ≤ 2.0; and in very poor condition, if CR < 2.0. Bresler, Okada and Zisling commented that the algebraic formulation as given in [20] is arbitrary, and that too much weight is given for present condition and too little weight is assigned to quantity rating.

Bertero and Bresler [8] presented damageability criteria according to local, global, and cumulative damage using the summation operation. An importance factor is introduced for each element depending upon such considerations as life hazard and cost.

Okada and Bresler [61] discussed the screening method, in which the reinforced concrete buildings are classified according to three types of failure mechanisms (bending, shear and shear-bending) by considering nonlinear response of the structure to two levels of earthquake motion (0.3 g and 0.45 g). The "first screening" deals with approximate evaluation of the load-deflection characteristic of the first story or of the weakest story. The "second screening" consists of a time-history nonlinear response analysis of each story. The "third screening" makes use of a dynamic response analysis including the nonlinearity of each member.

Recently, a safety evaluation program has been developed [51]. Subjective evaluations are obtained for exposure, vulnerability, and combined safety index. A digital scale of 0 through 9 is used with 0 denoting non-impact and 9 denoting severe impact. Weighting factors are applied to obtain a combined index for safety evaluation.

3.3 Application of Structural Identification Techniques to Damage Assessment

During this past decade, techniques of system identification have been successfully applied to solve structural engineering problems. Responses of a real structure to known forcing functions can be recorded and then analyzed to estimate the unknown parameters in a preassumed mathematical model. Although the resulting representation for the structure is an idealized model, it becomes more realistic than any "a priori" representations. The structural response to various expected loading conditions can then be computed using such a mathematical model for damageability evaluation or damage assessment.

In addition to using system identification techniques in obtaining the mathematical equation of motion for the structure, attempt can be made to directly assess the present damage level in existing structures. As an example, full-size structural members and connections have been tested under reversed plastic deformations [e.g.,2,52,62,65,82]. If the behavior of these full-scale specimens at various damage levels can be identified with the use of available techniques of system identification, a methodology may be established for the direct estimation of damage level of structural elements and thus of existing structures.

A virgin structure immediately after completion of construction, can be assumed to have an initial damage level, $d(t_0)$, on some scale, which may be caused by poor workmanship, inferior quality of materials used, or accidental loading conditions during construction. On the other hand, the total collapse of a structure can be assumed to correspond to a damage level of unity, which serves as the reference value on this damage scale. The damage of a structure can be indicated by (a) visually observable physical changes such as can be indicated by initiation and propagation of cracks or progressive failure of structural components, (b) directly measurable physical changes such as permanent or plastic deformations, (c) changes in abstract structural characteristics such as the damping coefficients, (d) change in mathematical modeling required to describe the behavior of the structure (e.g., the necessity of using nonlinear models for adequate representation indicates an advanced damage level. Lacking a precise understanding and thus definition of structural damage at present, it is necessary to make use of as many of these damage indicators as is practical and economically feasible.

For our purposes, the structure can be divided into major components (structural elements such as connections and members), each of which can be subdivided into localized points (macroscopic behavior of materials). At each level, there can be separate damage scales corresponding to the normalized local and global damage indices as suggested by Bertero and Bresler [8]. More generally, the methods of Wiggins and Moran [89] and Culver et al. [20] can also be summarized in a similar manner.

Various kinds of nondestructive tests can be conducted on the structure. Such test data can be used to estimate the appropriate damage level(s). For example, results of ultrasonic and/or X-ray tests are effective in detecting cracks and thus can be used in estimating the damage of localized points. The damage at this level thus estimated can be used for correlation with the damage level of structural elements and that of the whole structure, which can also be estimated directly or indirectly using results of other types of tests and/or observations.

Alternatively, various tests can be conducted to estimate the current (residual) values of strength, ductility, damping (energy absorption capacity), stiffness, and continuity. On the basis of these data, the overall structural damage may be estimated. Each of these quantities can be evaluated at several levels. For example, it is of interest to assess the continuity between (a) structure and foundation, (b) member to member, and (c) point to point.

3.4 Discussion

The ultimate objective of making damage assessment and damageability evaluation is to decide on necessary measures for hazard abatement [e.g.,12]. Recently, a suggestion was made to attempt the assessment of structural reliability as well [92]. The possible application of such a methodology to nuclear structures was discussed recently [94].

An important step in establishing such a methodology is to obtain a practical and unified definition of damage for various types of structures as well as for different scales of structural elements. Moreover, it is desirable to study the inter-relationships among damage from one scale to another. As an example, it is possible to evaluate the damage in the form of a crack at a certain location of the wall by performing one or more non-destructive tests. It is then desirable to find the influence of this particular damage in this wall element to the damage level of the whole structure.

4. RELIABILITY EVALUATION OF EXISTING STRUCTURES

4.1 General Remarks

During these past three decades [29] much progress has been made in the theory and application of structural reliability [3,30,31,80]. At one end of the spectrum, various approaches have been proposed to formulate the so-called Level I reliability-based design codes [1,26,27] which resemble current codes with relatively simple design formulas. At the other end of the spectrum, the state-of-the-art approach includes the application of random processes [54,63,76,90], risk analysis [21,70, 88], and optimum design of structures [55,91]. These advanced studies add a new dimension to the practice of structural engineering in treating natural phenomena involving various degrees of uncertainty. Once again, most of the investigations conducted to date deal with idealized mathematical models. In 1975, Galambos and Yao [34] pointed out the need for more experimental work in developing new design codes.

All the mathematical analyses and experimental investigations prior to the construction of structures are certainly necessary, and continuing research and development in these areas is desirable. On the other hand, there exists a need to periodically analyze and assess the reliability of certain structures that have already been built and that can be subjected to hazardous loading conditions such as strong motion earthquakes and extreme winds. Such is the case of nuclear structures where loss of integrity can lead to dire public consequences.

The objectives of this chapter are to:

(a) formulate the problem of assessing the reliability of existing structures, and

(b) explore several possible approaches to the solution of this problem.

4.2 Problem Statement

The reliability of a structure is denoted by $L_T(t)$ and is defined as the probability that the useful life, T, of the structure will be at least t, i.e.,

$$L_T(t) = P(T>t) \tag{4.1}$$

Alternatively, this function can be expressed in terms of two random processes: namely, R(t) denoting the resistance (or capacity) of the structure, and S(t) denoting the applied force (or demand) on the structure as follows:

$$L_T(t) = P[R(\tau) > S(\tau);\ 0 \le \tau \le t] \tag{4.2}$$

If we let D(t) denote the damage of the structure at time t, the reliability function can also be given by:

$$L_T(t) = P[D(\tau) < 1;\ 0 \le \tau \le t] \tag{4.3}$$

For structures undergoing no maintenance work, the reliability function thus defined is a non-increasing function of time t. The mathematical calculation of such a quantity in general can be very difficult indeed [53].

It is well known [30] that the reliability function can also be expressed in terms of the hazard (or risk) function, $h_T(t)$, defined as follows:

$$L_T(t) = L_T(0) \exp \left[-\int_0^t h_T(\tau)\, d\tau\right] \tag{4.4}$$

Consider now the case of a specific structure. During its lifetime, several hazardous events occur. The problem to be considered herein is the estimation (or assessment) of the quantities h(t), D(t), or $L_T(t)$ at the present time t, the results of which can be used to guide the decision whether major maintenance and repair work are needed for this particular structure.

4.3 Possible Approaches

As is done in current practice, a structure can be tested with known forcing functions. Standard methods of system identification [28, 77] can be used to estimate various structural parameters such as natural frequencies and damping coefficients. If several levels of the excitation are used, any detectable changes in each parameter can be considered a measure of damage in the structure at the time of testing. In this regard, the random decrement signature [88], which results from bandpass-filtering the time-history and then averaging all time segments at a given constant initial value, was recently applied for the detection of possible deterioration in bridge structures [17].

Because a high degree of nonlinearity in structural behavior usually corresponds to a high level of loading, another indicator for structural damage is the demarcation between linear and nonlinear structural models [45]. Recently, the Wiener technique of nonparametric identification has been applied to the case of earthquake response data of a reinforced concrete building [57,84]. Whether the second (or higher) order nonlinear kernel is needed for modeling purposes can be an indication of structural damage.

In the case of reversed loading conditions, cumulative fatigue damage may result. This type of "damage" is also an abstract quantity, though evidence of its presence can be observed on the atomic or crystalline scale. Recently, large structural elements such as full-scale members and connections have been tested under reversed plastic deformation [39,65]. If the behavior of these full-scale specimens at various stages of damage can be "identified" with techniques available in system identification, a methodology may be established for estimating the damage level of existing structures. Several investigations relevant to such studies have been reviewed by Liu and Yao recently [56].

5. SUMMARY AND DISCUSSION

In this technical report, an attempt is made to summarize the state of the art of system identification as applied in structural engineering, damage assessment and reliability evaluation of existing structures. During these past two decades, various-techniques of system identifications have been applied for the solution of structural engineering problems. The available literature in this regard has been critically reviewed and summarized in tabular form, which should be useful to structural engineers in general.

To-date, relatively few engineers have specialized in damage-assessment and/or reliability evaluation of existing structures. Moreover, it is difficult to transmit and dissiminate such expertise which is based primarily on personal intuition and experience. Several recommended procedures for inspection and safety assessment are reviewed and summarized herein. It is believed that further development and improvement are possible and desirable. The possible incorporation of system identification techniques into damage assessment is also discussed.

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Table 1. Summary of Models and Methods in Structural Identification [15]

Models \ Methods		Time Domain Analysis: Normal Modes	Time Domain Analysis: Non-Normal Modes	Time Domain Analysis: Impulse Response	Time Domain Analysis: Moments Technique	Time Domain Analysis: Perturbation Method	Time Domain Analysis: Energy Method	Time Domain Analysis: Estimation & Statistical Technique	Frequency Domain Analysis: Normal Modes	Frequency Domain Analysis: Impulse Response	Frequency Domain Analysis: Frequency Response	Frequency Domain Analysis: Spectral Approach	Frequency Domain Analysis: Estimation & Statistical Technique	Frequency Domain Analysis: Statistical Technique	Frequency Domain Analysis: Estimation w/o Statistical Technique
Linear Models	Lumped Parameter Models	18 37 40 42* 67	4 44	42* 87	50	44	5 67	4 5 14 18 19 22 37 40 58 59 60 64* 66 77	7 41 42* 49 68 79	84	7 13 41 42* 46 47 68 72 75 79 87	35 36 41 46 47 72 74 79 83	7 13 19 35 36 46 47 72 79 81 84	83	41
Linear Models	Continuous Parameter Models							85	79			79			
Nonlinear Models	Lumped Parameter Models	/	/		50			22 23 24, 25# 32 60, 78x	/	57			57		
Nonlinear Models	Continuous Parameter Models	/	/						/						

#Bilinear model

xExponential model

*Identification of forces

Table 2. Summary of Available Information [15]

Ref. No.	Forcing function													System														Response										
		Deterministic												Linear										Nonlinear														
						Wind		Earthquake						CBM			LMM							LMM														
	Random	Sinusoidal	Sweep sine	Impulse	Arbitrary	Ambient	Moderate, strong	Ambient	Moderate, strong	Other response	Free vib.	Not clear	Noise	M	C	K	W_n	M	C	K	Aircraft	Others	Unparametric	Coefficients	Aircraft	Unparametric	CBN	A	V	D	A & V	V & D	D & A	A, V & D	Stress	Pitching degree α	Aircraft	Freq. response
4									X									X	X	X												X						
5						X		X										X	X	X								X	X	X								
7		X																X	X	X								X	X									X
13		X																	X											X'								X
14		X							A				WF						X	X								X										
18	X									D							X	X		X										X								
19	?											X					X	X		X										?								
22									A				X						X	X				X					X	X		X						
23,24 25		X							A		X		X											X				X	X	X		X	X	X				
32						?	X						W												X			X								X	X	
35	X					X		X					W				X		X									X	X	X								
36	X					X											X	X	X	X										X								
37												X	X							X										X								
40										D	X							R		R										X								
41								X	A										X									X	X	X								X
42*							X										X	X	X	X								X										X
44										X		X							X									X										
46		X				X		X									X	X	X	X								X										X
47		X	X	X									WF				X		X											X								X
49		X																X	X	X										X								
50	X	X	X										WF					X	X	X				X								X		X				
57						X		X	X				W													X		X										
58,59									X										X	X		X											X					
60						?	X						WF								X				X			X								X	X	
64*				X			X		A									X	X	X								X	X	X					X			
66									X	DV	X						X		X										X									
67									X										X											X								
68		X																X	X	X								X	X	X					X			X
72		X	X			X		X					SF				X		X									X	X	X								X
74		X																	X	X								X										
75	X	X		X													X		X	X										X								X
77						?	X										X		X	X										X						X		
78										?		X												R						?								
79	?					X								X		X	X		X									X										X
81	?			X									W				X		X									X										
83	X																X											X										
84						X		X	X				W										X					X	X									
85		X							D				F			X														X								
87					X								F					X	X	X			X					X	X	X								

A - Acceleration F - Free S - Sine noise ? - Not very sure
D - Displacement R - Random V - Velocity * - Identification of forces
W - White noise

Table 3. Pertinent Information in Detail [15]

Ref. No.	Experimental Example			Mathematical Model**	
	Excitation	Structure	Response	Excitation	Structure
4	Vibration of soil	Single story bldg., Nuclear reactor	D & V	Ground motion	LMM; M, C, K
7	Sinusoidal (steady state)	Overhung beam; 18 D.O.F.; 3 modes	11 Velocity responses	Sinusoidal or transient	LMM; [M],[C],[K]
13	Constant forcing vector, f = 9 sin ωt	2 D.O.F. system with non-proportional damping matrix; 6 D.O.F. system	D (freq. response)	Sinusoidal	LMM; C
14	$\ddot{X}_o(t)$ = sin ωt, 0 ≤ t ≤ 12 sec (5%, 0% noise)	Building, 2 D.O.F.	A (5%, 0% noise)	Ground motion or sinusoidal	LMM; C & K
18	Random bending, Random shear	Free-free beam (2 finite elements, EI & Wn) and Saturn V(EI & GA_S, 28 finite elements)	D	Random	LMM; K,M,W_n
19	Shock & vibration	Spring-mass chain; 2 D.O.F.; K	Eigenvalue	Shock & Vibration	LMM; K, M
22	Seismic acc.	3 story steel frame, 3 D.O.F.	D or V or both	Ground motion	LMM; linear (K,C) or nonlinear (a_1 to a_4)
23	Seismic acc. & external force or free vib.	1 D.O.F. Viscoelastic materials	D, V, A or D & V	Seismic, sinusoidal, triangular sine, or exponentially decaying sine	LMM; nonlinear integral model, a_1 to a_7
23,24	Seismic acc. & external force or free vib.	1 D.O.F. shear frame	D, V, A or D & V	Seismic, sinusoidal, triangular sine, or exponentially decaying sine	LMM; nonlinear differential model, a_1 to a_4
23,25	Seismic acc. & external force or free vib.	1 D.O.F. steel frame, kinematic type elasto-plastic behavior	D, V, A or D & V	Seismic, sinusoidal, triangular sine, or exponentially decaying sine	LMM; bilinear hysteric model, a_1,a_2,a_3
32	Wind tunnel test	6 D.O.F. (or two 3 D.O.F. subsystems) aircraft	Linear angular acc., deg., etc.	Wind	Nonlinear LMM
35	Random winds	9 story steel frame budg.; 2 modes	D, spectral density (steady state)	Random winds or earthquake	LMM; C, W_n, period
37	Shaking	Cylindrical nonuniform beam, 0.05" aluminum; EI; 2 modes	D	Shaking	LMM; K or extended to M & K
40	Static axial member force	Linear statistical model, two bar stressed truss; random mass & elastic stiffness	D	Static axial load, initial displ.	LMM; M & K
41	Earthquake	Steel and concrete building; 3 or 5 modes	D, A or spectral V	Ground motion, ambient acc.	LMM; C
44				Driving force or other response point on the structure	LMM viscously damped system; C_{jj} and C_{jk}
46	Sinusoidal & ambient vib.	Buildings; eigenfrequency & damping; 3 modes	A	Sinusoidal, ambient	LMM; W_n, C, M, K

**table continued on next page.

Table 3 continued

Ref. No.	Response	Assumptions	Information: Incomplete	Information: Complete	Methods of Analysis
4	D & V	Damping isn't proportional. Phase information is used and good initial estimates of parameters are available.		X	M, E & T
7	V or A	D.O.F. of model > no. of measurable modes. Damping with a rather restrictive form. Measured modes are forced to be orthogonal to the unknown symmetric mass matrix.	X		M, E & F
13	D or freq. response	The physical system behaves exactly as modeled by equation. The measurements of the response vector $\bar{x}(w)$ are affected by zero errors. Knowing K & M.		X	E & F
14	A	Given mass matrix.			E & T
18	D		X		M, E & T
19	Eigenvalue & eigenvector	Required to make a linear approximation of the partial derivatives of measured quantities with respect to unknown parameters {R}.	X	X	E & T
22	D or V or both	The same unknown vector was assumed for each floor.			E & T
23	D, V, A or D & V	P(t) and M are assumed to be known.			E & T
23,24	D, V, A or D & V	P(t) and M are assumed to be known.			E & T
23,25	D, V, A or D & V	P(t) and M are assumed to be known.			E & T
32	Responses of aircraft	The prior statistics are known. Angular displ. are small. Rigid airframe. Constant mass. Steady flight.			E & T
35	D, V or A				E & F
37	D	Knowing M. The rotary inertias are zero.		X	M, E & T
40	D			X	M, E & T
41	D, A or spectral V	Damping to be of the proportional form. Limited to low response amplitudes.			M, E & F
44	A (co & quad)	Complete mass matrix. Small perturbation terms $\delta\phi_I$ are measurable. Considering the coincident & quadrature response of one pt..		X	M & T
46	A				E & F

Table 3 (cont'd)

	Experimental Example			Mathematical Model **	
Ref. No.	Excitation	Structure	Response	Excitation	Structure
47				Transient impulse, swept sine, sinusoidal	LMM; W_n, C
49				Sinusoidal	LMM; M, K, D
50	Random excitation	Coupled oscillators, one or two-dimensional chain-like system; K, C	D & V	Random, sinusoidal, sweepsine	LMM; M, C, K (linear or nonlinear)
57	Strong ground shaking, ambient vib. test	Nonlinear feedback system, R.C. structures	A	Ground motion, ambient	Nonlinear unparametric model; symmetric kernel $h_n(\tau)$
58,59	Earthquake or shaking table simulate earthquake	A single story steel frame	D & A	Ground motion	LMM; K, C, A, R
60	Wind tunnel test (w or w/o gusts)	HL-10, M2/F3 and nonlinear X-22 VTOL	Lat. acc., roll deg. & rate, etc.	Wind	LMM (linear or nonlinear); aerodynamic parameters
66	Earthquake, Free vib.	29-story budg. 4-story structure; W_n, ξ	Root-mean-square absolute vel. responses	Ground motion, initial displ. and vel.	LMM; W_n, C
67	Period ground motion	R.C. frame; ξ; 3 modes	D	Ground motion	LMM; C
68	Sin ωt, $0 \le \omega \le 60\pi$	1/10 & 1/40 scale Apollo/Saturn V; 3 or 4 modes; $2\mu\omega$, ω^2 & 1/m	D(1/10), A(1/40) & θ	Sinusoidal	LMM; M, C, K
72	Wind or microseismic (sine noise)	Large structures; ξ, W_n	D, V, A or freq. response	Ambient ground motion or wind	LMM; C, W_n
74	External & internal forces	Electrically powered railroad car, motor of a tracked air cushion vehicle	Lat. & vertical acc.	Sinusoidal	LMM; C, K
75	Forces & torques or rotating unbalance	Machine tool	D	Sine vib., pulse, random signal	LMM; C, K, W_n
77	Wind tunnel test & current field experiment	Scaled suspension bridge model; 1 & 2 D.O.F.	Heaving & pitching response	Wind	LMM; K, C, W_n
78	Initial displ.	A two-compartmental random model. (pharmaco kinetics)	Kinetic data	Initial displ.	Random exponential model; random rate constants
79	Wind vel.	Tall budg.	D & A	Ambient wind	LMM & CBM; W_n, C, K(z), M(z)
81	Impulse of random amplitude	Single D.O.F. model, shaken table; ξ, W_n	A	Impulse	LMM; C, W_n
83	Constant speed	Car model (complex sheet metal struc.); amplitude, phase of various pts.	Acc. power spectra	Random signal	LMM; modal frequencies, mode shapes

**table continued on next page

Table 3 continued

Ref. No.	Response	Assumptions	Information Incomplete	Information Complete	Methods of Analysis
47	D or freq. response				E & F
49	D (steady state)			X	M & F
50	D, V & A	Errorless measurements were obtainable.		X	S & T
57	A	The system is nonlinear but time-invariant over the time period during which the identification is carried out.			E & F
58,59	D & A	P(x) is a linear equation for a linearly elastic material.			E & T
60	Response of aircrafts				E & T
66	Filtered response vel.				E & T
67	D	Requires mode shapes & the frequencies of the structure & the energy ratio for various budg. elements. Low-amplitude nondamaging motion is assumed.		X	M, E & T
68	D, V, A or stress (steady state)	Light damping. Linear behavior. Widely separated modes.		X	M & F
72	D, V, A or freq. response				E & F
74	A (steady state)	Smooth-riding tracked vehicles for high-speed mass transit systems.			F
75	D or freq. response				F
77	D & deg.				E & T
78	D	Assuming that the initial conditions are deterministic constants. The distributional classes, e.g. normal, gamma, or beta of the rate constants are known.	X		E & T
79	Acc. response spectrum				M, E & F
81	A	Needs second-order differential eq. Noise is additive; normal, band-limited from response to response. Sample rate is sufficient to eliminate aliasing errors associated with the system response and the noise.			E & F
83	Acc. power spectra			X	S & F

Table 3 (cont'd)

Ref. No.	Experimental Example			Mathematical Model **	
	Excitation	Structure	Response	Excitation	Structure
84	Earthquake, ambient	9-story R.C. structure	V or A	Ground motion, ambient wind or microtremor	Linear unparametric model; symmetric kernel $h_n(\tau)$
85	A sum of 4 sinusoids or component of ground displ.	A continuous shear beam	D	Ground motions, sinusoidal	CBM; k(x) or extended to k(x) & m(x)
87	Arbitrary acc.	Undamped simple oscillator; $h_{ij}(t)$	A	Arbitrary	LMM; $h_{ij}(t)$ or M, C & K
5	Force vector	9-story steel structure	D or V or A	Ambient or forced excitations	LMM; M, C, K
36	Random wind	Natural frequency and damping	D	Ambient excitations	LMM; M, C, K

A - Acceleration
D - Displacement
E - Estimation method
F - Freq. domain analysis
M - Modal method
S - Statistical technique
T - Time domain analysis
V - Velocity

**table continued below

Table 3 continued

Ref. No	Response	Assumptions	Information		Methods of Analysis
			Incomplete	Complete	
84	V or A				E & F
85	D	The mass m(x) per unit height of the structure is known. Measurements are noise free and comprise of the time histories. Structure starts from rest.		X	E & T
87	D, V or A	Input & output data are noise free.		X	F or T
5	D or V or A	The mass or stiffness matrix need not be symmetric or positive definite. Mode shape need not be normal. Damping need not be proportional.		X	M, E & T
36	D				E & F

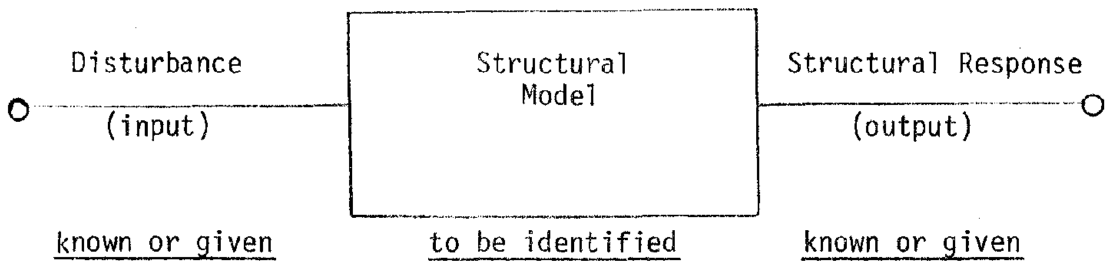

Figure 1. Structural Identification.

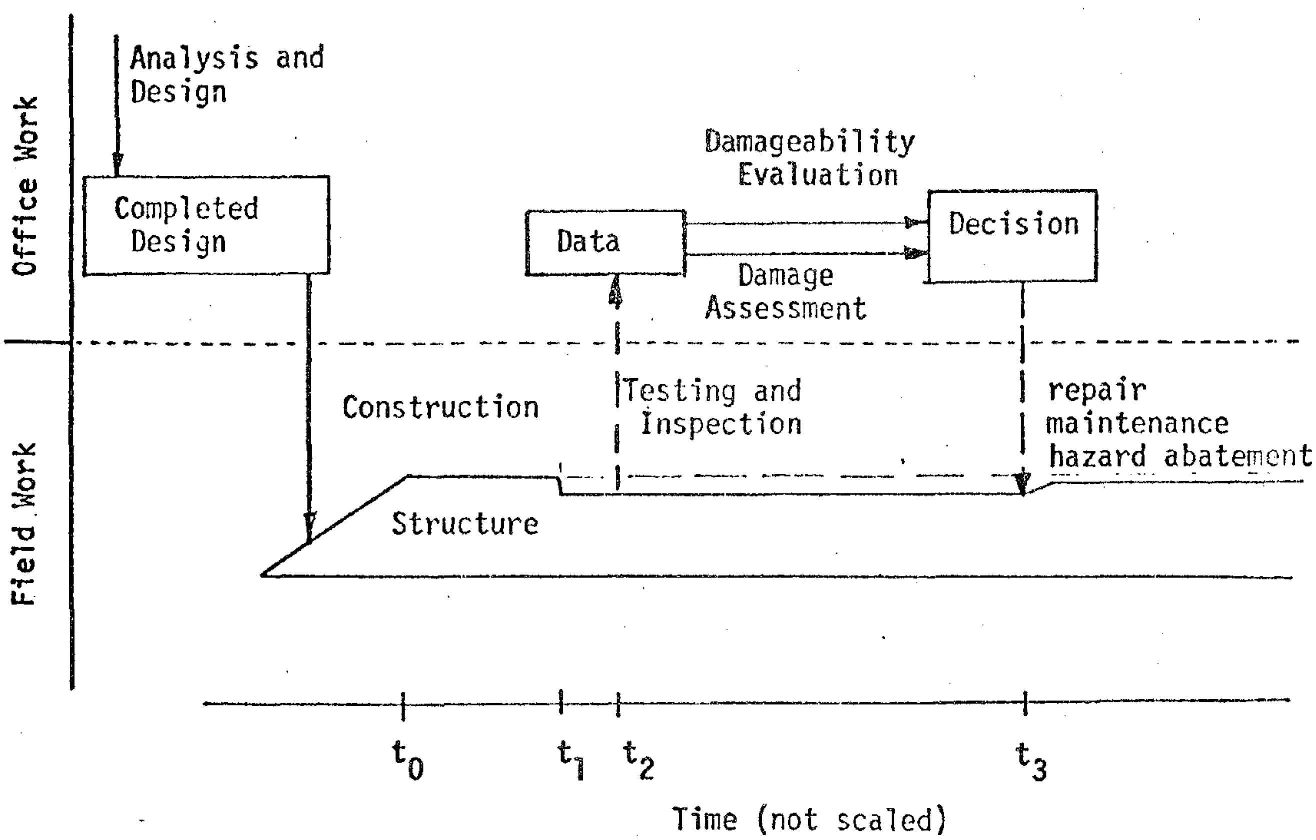

Figure 2. Roles of Damage Assessment and Damageability Evaluation During the Lifetime of a Structure [93].