IMPORTANCE OF MEASURED FULL-SCALE BUILDING RESPONSE IN THE COMPUTER MODELING OF BUILDINGS

by

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ABSTRACT

This paper presents a perspective on the importance of developing accurate computer models of constructed buildings and the role that measured earthquake response records play in advancing this area of structural engineering research and practice.

INTRODUCTION

I wish to express my sincere appreciation to the Strong Motion Program of the California Division of Mines and Geology for inviting me to speak on this most important topic. This part of my research has spanned almost 30 years and I have many, many special memories. One such special memory was doing research relating to developing a computer model for a high-rise building in San Diego. This research on full-scale response started in 1969 when I was a new UCLA Assistant Professor. I helped drag the cables up and down the San Diego Gas and Electric Building as my leaders, the late UCLA Professor R.B. "Fritz" Mathieson and Professor Paul Jennings of Cal Tech, vibrated the building with shakers and measured the building's natural periods of vibration, damping, and mode shapes.

Perhaps the single most important reason for measuring full-scale building response is that it records the real motion of the building, and therefore, documents what really happened and not just what the computer model says happened. I remember as if it were yesterday, walking up and down the stairs of the San Diego Gas and Electric Building as it was being vibrated at one of its higher mode natural frequencies. I could feel the decrease in motion as I moved toward the nodal points in the mode shape and the increase in motion at the locations of greatest modal displacement. This was a real high-rise building and it really did have mode shapes!

I learned in the 1970s from Professor Ray Clough of UC Berkeley that the purpose of an experiment is to serve as a reality check, or verification, of a computer model of the structure being tested. Therefore, the past and future acquisition of measured full-scale response of buildings and the study of this response is an essential part of the development of performance based design criteria. The need for accurate computer models of buildings that are used by structural engineers has never been greater than it is now. The economic loss incurred by the urban community during the Northridge earthquake was not acceptable. Unfortunately, the computer models that are used by most structural engineers for designing buildings do not have a high credibility rating when they are used to estimate building performance. Without a strong consensus on the risk and probable loss in future earthquakes based on good computer models of buildings, the "stoppers" within the business and structural engineering community will block any serious spending of money now to prevent future loss of life and dollars. My hope is that this paper will present some insight into the importance of measured response and the barriers that must be removed to help utilize the full potential of measured full-scale building response.

THE EVOLUTION OF COMPUTER MODELING OF BUILDINGS

The evolution of the computer modeling of buildings and other civil structures can be viewed as a journey down a very long and winding "technical load path." Imagine the following sections of this path:

Section 1: Buildings and bridges needed to be built (e.g., roman time) and unfortunately no mathematics tools or IBM Aptiva computers existed to develop a computer model of the structure. Therefore, without the benefit of a stiffness method of analysis or the Newmark-β numerical solution routine, the focus of the structural engineer was on the loads imposed on the structure (e.g., 200 soldiers on a bridge). The attempt by the structural engineer was to address the reality of the loading on the structure and then to develop a "structural design" for a building based on "professional experience" and hope that the building does not fail in the next large load experience.

Section 2: A beautiful baby called applied mathematics was then born and this birth was followed by the birth of children, grandchildren, and great-grandchildren that all became members of a structural engineering family called Structural Mechanics. The early members of this structural mechanics family (e.g., pre-1950s) helped develop analytical models of structures that followed a systematic and traceable computational path. However, this development was only possible after the structural engineer made the assumption that the materials were linear and the deformations were small. Therefore, the structural engineering world transformed itself from the reality of the loads to the new and improved world of linear elastic analysis. The flaw in this new world was that when the best estimate of real loads was applied to this new world linear elastic building model, the result was that the element deformations that were calculated exceeded the elastic limit of the element. What to do? The approach that was followed by the structural engineer was simple and it was to use a "fake" set of loads and earthquake ground motions. These "fakes" were loads much smaller than real loads, but they were winners because they produced element response in the linear elastic response domain. This approach was justified on the grounds that we really do not know the real loads on the building with any real accuracy. It also was justified on the grounds that if we did not use this new linear elastic approach we would be "technical dinosaurs." Of course this entire linear elastic world expanded beyond the sphere of civil and military structures when the aerospace program and the cold war started in the 1960s. Spacecraft and airplanes were expected to respond to their real loads in the elastic response domain and thus the analysis made sense. However, buildings are not expected to response to "the big one" in the elastic response domain.

Section 3: The desire to understand and use real loads and earthquake ground motions emerged in the 1970s like spring flowers. During this time, earthquake ground motion recording units were introduced at first on the ground and then later in buildings. The extra special efforts of the USGS, the Building of Safety Department of the City of Los Angeles, and the Strong Motion Program of the CDMG, have brought Structural Engineers back to where their focus is on trying to address the reality of the loads and the real earthquake ground motions. The refocus on loads and the climb up the real world learning curve started with the recording of a few relatively strong earthquake time history records on the ground away from all structures. Then the 1972 San Fernando earthquake produced records of motion at numerous ground locations throughout southern California and also in several buildings. More recently the Whittier, Loma Prieta, and Northridge earthquakes have provided many records of earthquakes ground motion and building response. The existence of these full-scale building response records has made clear the real need to develop an accurate model of the real building. A linear elastic analysis alone just does not cut it! Therefore, nonlinear computer models of buildings must be developed and these computer models must seek to estimate the response and performance of the specific building that is the focus of the attention of the structural engineer to the site specific ground motion description that is provided by the Geotechnical Engineer. Of course, the entry of the computer age (i.e., the PC age, not the mainframe age) has offered computational tools that could not have been imagined by most structural engineers just a few years ago. However, when all is said and done, the new world is one where for a particular new or existing building, the Structural Engineer and the Geotechnical Engineer work together as a team to develop and use the best real estimates of earthquake ground motions with the best possible computer model of the real building, to estimate the performance of the building and its probability of responding beyond defined performance limit states.

LINEAR ANALYSIS MODELS

Performance Based Design requires computer models of buildings that are both linear and nonlinear. Consider first the two types of linear analysis models.

A **RESPONSE SPECTRUM COMPUTER MODEL** is today the most common building computer analysis model used by structural engineers. This type of computer model is really a static response model because the response is not calculated as a function of time. However, this model has its foundation in structural dynamics and specifically with an analysis procedure called the Normal Mode Method. In the normal mode model analysis procedure, the matrix equation of motion of the building system in its physical coordinates, $\{X\}$, are transformed into a set of coordinates called normal mode coordinates, $\{q\}$. These normal mode coordinates when used transform the matrix equation of motion into a matrix equation with only diagonal matrices. To show this, consider the equation of motion in the physical coordinates as

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K]\{X\} = -[M]\{I\} / a(t) \qquad (1)$$

where

[M], [C], [K] = mass, damping, and stiffness matrices

$\{X\},\{\dot{X}\},\{\ddot{X}\}$ = displacement, velocity, and acceleration vectors for motions relative to the ground

a(t) = earthquake ground motion acceleration

If the building has *n* degrees-of-freedom, then the second order matrix linear differential equation of motion given by Equation (1) represents a coupled set of *n* differential equations because one or more of the matrices [M], [C], and [K] are non-diagonal.

If a special set of new coordinates, called normal mode coordinates, are defined by the equation

$$\{X\} = [\phi]\{q\} \qquad (2)$$

where

$\{q\} = (q\,(t)\; q_2\,(t)\; \ldots\; q_m\,(t))^T$ = normal mode coordinate vector ($m \le n$)

$q_i(t)$ = i^{th} normal mode coordinate

and

$[\phi]$ = transformation matrix from the $\{X\}$ to the $\{q\}$ coordinates

Then it follows if $[\phi]$ is a very special matrix that Equation (1) becomes

$$\left([\phi]^T[M][\phi]\right)\{\ddot{q}\} + \left([\phi]^T[C][\phi]\right)\{\dot{q}\} + \left([\phi]^T[K][\phi]\right)\{q\} = [\phi]^T[M]\{I\}a(t) \qquad (3)$$

Now if we define

$$[M^*] = [\phi]^T[M][\phi]$$
$$[C^*] = [\phi]^T[C][\phi]$$
$$[K^*] = [\phi]^T[K][\phi]$$
$$\{G\} = [\phi]^T[M]\{I\}/[M^*]$$

then

$$[M^*]\{\ddot{q}\} + [C^*]\{\dot{q}\} + [K^*]\{q\} = -[M^*]\{G\}a \tag{4}$$

The very special matrix $[\phi]$ is called the Modal Matrix. Each column of this matrix is a vector and is called a building mode shape of vibration. The first column of $[\phi]$ is called the first mode shape of vibration, denoted $\{\phi\}_1$, the second column of $[\phi]$ is called the second mode of vibration, denoted $\{\phi\}_2$, etc. When the model matrix is used to define the coordinate transformation from the $\{X\}$ to the $\{q\}$ coordinates, then it can be shown that [M*], [C*], and [K*] are all diagonal matrices. Therefore, the building response in the normal mode coordinates, $\{q\}$, can be calculated by solving up to *n* uncoupled second order differential equation of motion. Equation (2) can now be written as a summation, or power series expansion, of the response $\{X\}$ in terms of the $\{q\}$ coordinates. Equation (2) becomes

$$\{X\} = [\phi]\{q\} = \{\phi\}_1 q_1(t) + \{\phi\}_2 q_2(t) + \ldots + \{\phi\}_m q_m(t) \tag{5}$$

Each uncoupled equation of motion is of the form

$$\ddot{q}_i(t) + 2\xi_i\omega_i\dot{q}_i(t) + \omega_i^2 q_i(t) = -G_i a(t) \tag{6}$$

where

G_i = participation factor for the i^{th} mode and i^{th} element in $\{G\}$

$$= \{\phi\}_i^T[M]\{I\}/[M^*] \tag{7}$$

and

$\{\phi\}_i$ = i^{th} mode shape of vibration of the building

The response of the building is calculated by first representing the acceleration of the ground, *a(t)*, by a function called a Response Spectra, $S_a(T)$, which incorporates the amplitude and frequency response characteristics of *a(t)*. The function $S_a(T)$ is obtained by calculating the response of a series of single degree-of-freedom systems with varying natural frequency of vibration and modal damping. With a knowledge of $S_a(T)$ the structural engineer can develop a computer model of the building and then calculate the maximum response of q_i, or any associated building force or displacement, for i = 1,2,..., m where m is less than or equal to n. In this response solution, the maximum value of the response is known, but the time at which this maximum response occurs is not retained and thus not used in the analyses.

When this computer model is used to estimate building performance the natural frequencies of vibration, ω_i, modal damping values, ξ_i, and mode shapes $\{\phi\}_i$ are the critical structural modeling parameters. It is very important when using this method to have very accurate estimates of these modeling parameters. Measured full-scale building response is essential in developing models that can be used with confidence when the structural engineer performs a Response Spectra Analysis.

A **LINEAR DYNAMIC TIME HISTORY COMPUTER MODEL** calculates the loads on the structural members from the earthquake, dead and live loads, and compares these loads for each structural member with the yield capacity of the structural member. Equation (1) can be solved directly using direct integration methods such as Newmark-β or Wilson-θ. Alternately, the normal mode method can be used and Equations (5) and (6) are used to calculate the building response. In both methods, the response is calculated as a function of time. Unlike the Response Spectra Method where the time variation of response is lost, here the values of all response quantities are calculated at each instant in time. In both of the above noted linear computer model analysis procedures, the ratio of the Load Demand to the Yield Capacity is termed a Demand-Capacity Ratio (DCR). The value of this ratio is taken as a measure of the inelastic (or ductility) demand on the structural element. A linear elastic time history analysis can provide reasonable estimates of the DCR's for ratios less than 1.5 or 2.0. The accuracy and the range of applications of these linear elastic methods are not well defined primarily because of the inability of a linear elastic model to modify the earthquake load path within the structural system as yielding occurs within the building's structural elements. This load path modification is especially important for brittle, non-ductile elements. Some creative ways have been proposed to soften the stiffness at the structural members as a function of response amplitude. The relationship between the axial load on the vertical structural elements (e.g., columns and walls) and the stiffness and capacity of these elements is not captured by a linear dynamic time history analysis. Measured full-scale response is essential to define the limits of this computer model and especially the DCR limits.

NONLINEAR ANALYSIS MODELS

A **NONLINEAR STATIC COMPUTER MODEL** represents the variation of stiffness with deformation and thus seeks to improve upon the performance prediction accuracy of the linear analysis models. A major shortcoming of this model is that it does not directly model the time variation of building response. This type of model has the advantage that it can modify the load path of the earthquake loads within the structure as the structural elements yield. Also, by its nature it is a displacement-based procedure and the earthquake elements are in displacement-based response variables (e.g., curvature, strains, plastic rotations, etc.) and not forces. It is essential where real accuracy is desired to quantify building response from the deformation perspective because only by doing so can the structural engineer understand building performance. The inelastic deformation demands can be compared with the inelastic deformation capacities to determine the level of safety associated with any identified limit state.

A basic shortcoming of a Nonlinear Static Computer Model approach can be seen by examining the basic equation of motion in Equation (1) in its rearranged form

$$[K]\{X\} = -[M]\{\ddot{Y}\} - [C]\{\dot{X}\} \tag{8}$$

where

$$\{\ddot{Y}\} = \{\ddot{X}\} + \{I\}a(t) = \text{absolute acceleration}$$

In a time history analysis, the forcing function acting on the building at any instant in time is the summation of the inertia force, $\{F_I\} = -[M]\{\ddot{Y}\}$, and the damping force, $\{F_D\} = -[C]\{\dot{X}\}$. A static nonlinear analysis uses a static force, denoted here as {F}, that is intended to provide an acceptable approximation to $\{F_I\}$ and $\{F_D\}$ for the purposes intended by the structural engineer. The difficulty with developing a "good" {F} can be seen in Equation (8). In this equation, both $\{F_I\}$ and $\{F_D\}$ are functions of time. Therefore, to develop a "good" {F} the force vector must capture with one constant force vector, {F}, the influence that two time varying force vectors $\{F_I\}$ and $\{F_D\}$ have on the structure. This is very difficult to do even for modest response equivalency goals. Also the building responding in the nonlinear response domain results in the stiffness matrix [K] having a variation as a function of the building displacements and therefore time. However, despite these shortcomings, a static nonlinear analysis is preferred as a preliminary, relatively low cost and effective way to start to understand a building's performance. Full-scale building response measurements are needed to define the limits and the accuracy of this computer model approach.

A NONLINEAR DYNAMIC TIME HISTORY COMPUTER MODEL solves Equation (8) for one or more site specific earthquake ground motions and for one or more sets of possible parameter values. A Nonlinear Dynamic Time History Computer Model will always be more accurate than a Nonlinear Static Computer Model. The criticisms of a Nonlinear Dynamic Time History Computer Model are usually centered around budget limitations placed on the computer analysis phase of a project or the educational limitations of most structural engineers. Both of these limitations are probably true, sad, and inexcusable!

Nonlinear computer models are the most accurate and most valuable computer models. However, the verification of a computer model using measured full-scale building response is also the most difficult. There are several reasons for this. First, it is the authors experience that many structural engineers have a very difficult time learning the topic of linear structural dynamics. This topic is not easy to learn and requires a good understanding of applied mathematics. Second, the step from linear to nonlinear structural dynamics is also very difficult, and thus many structural engineers just do not start the learning process or give up. Third, as pointed out over twenty years ago by Professor Penzien and his Ph.D. student, Dr. Ruiz, the response of a building modeled using a nonlinear computer model is very sensitive to the values of the parameters in the model and the time variation of the earthquake ground motion. Stated another way, the closer we get to reality the more sensitive the solution is to the parameters of the model and the ground motion. A simple non-engineering example might help understand this point. Assume that you have never had a pair of shoes on your feet and you purchase your first pair. The shoes will feel great especially when walking across a southern California beach on a day where the temperature is 105°. Now time passes and you purchase shoes occasionally and each time you learn more about how to select shoes to make your feet more comfortable. For example, the size, width, vibration soles, arches, etc. Now the process of selecting the correct shoe for you has become more "scientific", more difficult and more time consuming. However, if you are patient and expend the effort your new shoes will fit better than your old shoes. The nonlinear computer analysis is like a new shoe and the linear analysis is like an old shoe.

It is essential to obtain measured full-scale response data to verify our nonlinear computer models. The more locations where the response is measured (both in the building and in the adjacent soil), the better our ability to understand and model the buildings nonlinear response.

The benefit cost ratio for structural engineers of recorded response measurements is very high. However, until recently the structural engineering codes of practice deterred the structural engineering from using improved computer building models and thus indirectly discouraged learning from records of building response.

LEARNING FROM EARTHQUAKES

On occasion the author has told the story of when he was a young, green Assistant Professor at UCLA in the late 1960s and he was attending an earthquake engineering conference. On Day 1 of the conference, the author had given one of his "passionate" talks about earthquake analysis of buildings and the need to incorporate the uncertainty in the computer model and also the uncertainty in the earthquake ground motion. On Day 2 of the conference, the author entered the conference restaurant for breakfast and he was asked by Professor Newmark to join him. Of course the author accepted and of course he shook in his chair all during the breakfast and the estimated peak acceleration was 2g. Little is remembered of that breakfast besides the honor of the invitation and one very wise piece of advice from this great structural engineering professor. What Professor Newmark said was "Things do not change fast in the area of building earthquake engineering so have patience."

The author now, at the age of 50+ and very near the age that Professor Newmark was that day, understands the keen insight in this advice. The author now understand after serving for over 15 years as a member or observer on the SEAOC Seismology Committee and learning from real earthquake experiences many of the justifiable reasons for the slow changes in the accepted use of structural engineering computer modeling methods. The only real time we have to observe, record, and learn from nature is during an earthquake. The single most important item that a structural engineer can have to maximize the learning from the earthquake is to have a measured response of a building. Therefore, the slow change is primarily the result of a lack of good quality measured building response records combined with the severe consequences that having a wrong model can have on the lives of hundreds or even thousands of people in a building.

VISION 2000 AND COMPUTER MODELING

Consider the quotation from the Vision 2000 part of the SEAOC Blue Book: "A major challenge to performance-based engineering is to develop simple yet effective methods for designing, analyzing, and verifying the design of structures so that they reliably meet the selected performance objectives." This quotation identifies perhaps the single largest reason for the development and use of realistic computer models of buildings. The reason is a lack of respect for the complexity of the problem being solved by the public and structural engineers. Even more, this comment shows a lack of professional respect for the desires and capabilities of Structural Engineers.

Structural Engineering Design is in many ways very similar to the design process that a trained artist uses when designing a painting. Both involve a blend of past life experiences, available technology, and subjective personal feelings. I perform calculations when I design a building until I have a comfort level that tells me that the design is done and the performance goals that have been defined have been satisfied. My own personal line is "That is enough analysis, the design feels right." Design is a process where analysis creativity and effort are at the direction of the structural engineer and the analysis has only one goal and that is to provide the structural engineer with this level of comfort in the design.

To many who have not stamped the plans for the design of a tall building the comments in the preceding paragraph may seem like a bunch of hot air. Also, to many who do not want to do any computer modeling of buildings this many seem like a quick way out. And to many who do not model real buildings on the computer this may seem like an excuse to not perform a "more exact" computer model. However, it is none of the above. It is a realization that we must always push forward our abilities to accurately model real buildings on the computer but we must also recognize the realities of structural engineering.

A major challenge that Performance Based Design faces is the real but well disguised resistance by many, many structural engineers to developing a good analytical model for the building. There are several reasons for this very strong resistance such as: (1) Current staff members are not able to do such analyses and as such must be trained or new staff employed; (2) The current structural engineering value system is often not compatible with rewarding engineers that are good analysts but are not as good at holding a clients' hands; (3) The scope of the required analysis for a project often changes as more is learned about the building and its performance. This often necessitates more effort and the request for more fees; (4) By its very nature computer analysis very often has hidden traps (e.g., bugs, wrong input, etc.) and thus the time line of work is more difficult to predict and control; and (5) Many senior engineers who are really small business operators and who do not really desire to understand how their building performs, see analysis as a very negative cash flow item. Performance Based Design addresses damage and serviceability limit states.

The Vision 2000 document identifies for seismic risk hazards, see Table 1.

The types of computer models of buildings has to be influenced by the different anticipated performance levels. Vision 2000 identifies four performance levels and they are:

- **Fully Operational.** The building continues in operation with negligible damage.
- **Operational.** The building continues in operation with minor damage and minor disruption in nonessential services.
- **Life Safety.** Life safety of the building occupants is substantially protected, but the damage is moderate to extensive.
- **Near Collapse.** The structural collapse of the building is prevented. However, life safety is at risk and the damage is severe.

Tables 2 to 5 expand on the above descriptions. These different performance levels place a special emphasis on obtaining full-scale building response records over a spectrum of amplitudes of motion.

Typical building response parameters are stress ratios, drift and deformation ratios, structural accelerations, ductility demand ratios, and energy dissipation demand vs. capacity.

The Vision 2000 document states: "Typical limiting values for these response parameters must be established for each performance level through research that includes laboratory testing of specific components, as well as calibrating the limiting values by <u>analyzing buildings that have experienced measurable damage in past earthquakes</u>." (The underline is the authors).

Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability. There are multiple sources of uncertainty in the seismic design process. There is uncertainty regarding the seismic ground motions, the design and analysis techniques and modeling assumptions, the variability in the materials, workmanship and construction quality, and the changes to the structure over its life due to material deterioration, wear, and structural or nonstructural modifications.

It is an important task of performance-based engineering to recognize, identify, and quantify those uncertainties using computer models of the structure so that the levels of reliability and risk can be established and acknowledged by the design professional, the client, the legal profession, and the public.

THE 1997 UBC AND COMPUTER MODELING

Some progress to reward more detailed computer models of buildings has been made in the last decade. For example, the 1997 UBC recognizes the improved accuracy of a response spectra analysis versus a static equivalent lateral force analysis. The 1997 UBC allows the elastic response parameters (i.e., design values) that are considered in a response spectrum analysis to be reduced for regular structures to not less than 80 percent of the basic base shear value from a static lateral load analysis.

The 1997 UBC states that: (1) Nonlinear time history analysis shall use site-specific ground motion time histories, (2) Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, (3) The maximum inelastic response displacement shall comply with standard code limits (i.e., 1.5%), (4) A design review of the lateral force resisting system shall be performed by an independent engineering team.

It is also very important to note that the Engineer of Record is required to submit with the plans and calculations a statement by engineering review team stating that the review included the items in the code.

The Base Isolation Design Criteria in the 1997 UBC has provisions that provide a reward to the owner when more detailed analytical computer models are developed and used in the design of the building. These rewards are incentives for the development of more accurate models and are important for inclusion into the building code.

LIMIT STATE DESIGN AND COMPUTER MODELING OF BUILDINGS

Without the reward to the owner of a positive benefit to cost ratio, most owners will <u>never</u> pay the structural engineer for the time required to develop a good analytical computer model of a building. Therefore, because of this belief the author has pushed for over a decade for first the inclusion into design of limit state design concepts, and then the inclusion of capacity reduction factors that are based on structural reliability concepts that quantify the uncertainty in the performance of the designed building. To illustrate the benefits of such an approach and how measured full-scale building response data provides a very strong financial reward to the citizens of California, consider the following discussion.

Consider a building located in Los Angeles and assume that the NEHRP regulations and maps are used to calculate: (1) the earthquake ground motion, (2) the static lateral force at a floor level, and (3) the equivalent elastic rotational demand on the member using an ETAB computer model. Also assume that the failure of the connection is at a plastic rotation of 0.005 radians. Will a specific connection fail? Your answer <u>must</u> be yes or no. You then must quantify how confident you are in your statement. For example, you might reply that you are "70% sure that this connection will fail in this building in the next 10 years." Your confidence will depend on such factors as how much time you spent on constructing your computer model (two days vs. 10 days); how thoroughly you checked out your computer model for input errors (5 minutes vs. 2 days); the type of welding done for the connections; whether the question was asked one day, one year, or one decade after the Northridge earthquake (i.e., the state of knowledge); etc.

Now if you answered the same question after a site-specific hazard analysis by a geotechnical engineer; the development of a nonlinear computer model of the building that included load redistribution because of structural yielding; the performance of a reality check of your computer model of the building using a previously recorded measurement of the building response; non-destructive testing of 30% of the connections in the building; etc. It is clear that even if the answer is the same, i.e."70% sure that this connection will fail in this building in the next 10 years," the more scientific analysis is, more credible and it should have a much greater weight. That is: Would I accept an answer from the latter method? Yes. Would I accept and answer from the former method? No. What if the former method shows not a 70% but a 99% probability -- would I accept it? Maybe.

A specific limit state that is being addressed as part of the design and rehabilitation of steel frame buildings will now illustrate the benefits of accurate computer modeling of buildings and measured full-scale building response. Imagine that a moment resisting steel frame is being designed. Also imagine that the limit state under consideration is the plastic rotation in the beam just adjacent to the beam-column connection. The limit state equation is

$$\begin{aligned} F &= \text{Limit State Variable} \\ &= \theta_c - \theta_d \end{aligned} \tag{9}$$

where

θ_c = capacity of the beam-column connection under earthquake loading

θ_d = earthquake induced demand on the beam-column connection

If the demand exceeds the capacity, the connection is defined to have failed. That is

$F > 0$ Safe Behavior

< 0 Failure

The mean value of the demand and capacity are denoted $\overline{\theta_c}$ and $\overline{\theta_d}$,respectively. Then the mean value of F is

$$\overline{F} = \overline{\theta_c} - \overline{\theta_d} \tag{10}$$

and the Central Safety Factor is

$$FS\ (\text{central}) = \overline{\theta_c} / \overline{\theta_d} \tag{11}$$

If θ_c and θ_d are considered to be independent random variables then the standard deviation of *F* is

$$V_F = \sqrt{V_c^2 + V_d^2} \tag{12}$$

where

V_c = standard deviation of θ_c

V_d = standard deviation of θ_d

The Safety Index denoted β for this limit state is

$$\beta = \frac{\overline{F}}{V_F} = \frac{\overline{\theta_c} - \overline{\theta}_d}{\sqrt{V_c^2 + V_d^2}} \tag{13}$$

In a rational world a value of the safety index would be set for a specific limit state depending on the consequences of not meeting the limit state (i.e., failure). For example, the more undesirable the failure the larger the value of the safety index. Equation (13) can be rearranged such that

$$\overline{\theta_c} = \overline{\theta_d} + \beta\sqrt{V_c^2 + V_d^2} \tag{14}$$

Assume for purposes of discussion that a simple non-scientific approach produced the same best estimate (mean) demand on the connection as the more sophisticated approach (i.e., the $\overline{\theta_d}$ is the same for both analysis methods). Therefore, the required average capacity is directly related to the square root of the sum of the standard deviation squared (i.e., variance) of the estimated capacity and demand. Assume for purposes of illustration that you had one half the uncertainty in the solution for the more scientific approach. Let $\sqrt{V_c^2 + V_d^2} = V_{ns}$ be the standard deviation of the non-scientific approach. Then the scientific approach would result in $\sqrt{V_c^2 + V_d^2} = 0.5V_{ns}$.

The required mean capacity for a non-scientific solution is

$$\overline{\theta}_{c,n} = \overline{\theta}_d + \beta V_{ns} = \left[1 + \beta\left(\frac{V_{ns}}{\overline{\theta}_d}\right)\right]\overline{\theta}_d \tag{15}$$

and the required mean capacity for the scientific solution is

$$\overline{\theta}_{c,s} = \overline{\theta}_d + 0.5\beta V_{ns} = \left[1 + 0.5\beta\left(\frac{V_{ns}}{\overline{\theta}_d}\right)\right]\overline{\theta}_d \tag{16}$$

If Equation (16) is divided by Equation (15), it follows that

$$\left(\frac{\bar{\theta}_{c,s}}{\bar{\theta}_{c,n}}\right)=\left[\frac{\left[1+0.5\beta\left(V_{ns}/\bar{\theta}_d\right)\right]}{1+\beta\left(V_{ns}/\bar{\theta}_d\right)}\right] \tag{17}$$

For convenience assume that $\left(V_{ns}/\bar{\theta}_d\right)$ is equal to 0.3 and $\beta = 3$, which are both reasonable assumptions. Then

$$\left(\frac{\bar{\theta}_{c,s}}{\bar{\theta}_{c,n}}\right)=\left[\frac{\left[1+0.5(3)(0.3)\right]}{1+(3)(0.3)}\right]=\frac{1.45}{1.90}=0.8 \tag{18}$$

Therefore, there is a 20% reduction in the acceptable value of the mean capacity associated with this modest increase in confidence. This can be translated into cost savings for construction or perhaps no seismic rehabilitation being required. This value for an increase in confidence could easily result if more measured full-scale building response data was available to help verify our computer model.

In the above example, it was assumed that the same average (or best) estimate was obtained using the non-scientific and scientific computer model. Human nature usually results in more conservative assumptions when there is a lack of sufficient full-scale test data. Therefore, the 1 in the numeration of Equation (17) would probably also reduce with the acquisition of more test data. For example, if the 1.0 were to reduce to 0.7, then Equation (18) will become (1.15 / 1.90) = 0.6 or a 40% reduction in the acceptable value of the mean capacity and an even more significant cost savings.

CONCLUSIONS

The measurement of full-scale building response is a very important part of the development of the performance based design of buildings. Computer models of buildings must be verified using actual building measurements. With the advancement of limit state design methods the reduction in uncertainty associated with a verified computer model has a direct positive impact on new or seismic rehabilitation building costs.

TABLE 1 VISION 2000 SEISMIC HAZARD LEVELS (SEAOC, 1996)

Event	**Recurrence Interval (Yrs)**	**Probability of Exceedence (%)**				
		10 yrs	20 yrs	30 yrs	50 yrs	100 yrs
Frequent	43	23	47	50	100	100
Occasional	72	14	28	42	50	100
Rare	475	2	4	6	10	20
Very Rare	970	1	2	3	5	10

TABLE 2 VISION 2000 DAMAGE BY PERFORMANCE LEVEL (SEAOC, 1996)

	Performance Level				
System Description	**Fully Operational**	**Operational**	**Life Safety**	**Near Collapse**	**Collapse**
Overall Building Damage	Negligible	Light	Moderate	Severe	Complete
Permissible Transient Drift	<0.2%±	<0.5%±	<1.5%±	<2.5%±	>2.5%±
Permissible Permanent Drift	Negligible	Negligible	<0.5%±	<2.5%±	>2.5%±
Vertical Load-Carrying Element Damage	Negligible	Negligible	Light to moderate, but substantial capacity remains to carry gravity loads	Moderate to heavy, but elements continue to support gravity loads	Partial to total loss of gravity load support
Lateral Load-Carrying Element Damage	Negligible. Generally elastic response; no significant loss of strength or stiffness	Light. Nearly elastic response; original strength and stiffness substantially retained; minor cracking/ yielding of structural elements; repair implemented at convenience	Moderate. Reduced residual strength and stiffness, but lateral system remains functional	Negligible residual strength and stiffness; no story collapse mechanisms, but large permanent drifts; secondary structural elements may completely fail	Partial or total collapse; primary elements may require demolition

TABLE 3 VISION 2000 FRAME DAMAGE BY PERFORMANCE LEVEL (SEAOC, 1996)

	Performance Level			
Elements	**Fully Operational**	**Operational**	**Life Safety**	**Near Collapse**
Concrete Frames	Negligible	Minor hairline cracking (0.02"); limited yielding possible at a few locations; no crushing (strains below 0.003)	Extensive damage to beams; spalling of cover and shear cracking (<1/8") for ductile columns; minor spalling in nonductile columns; joints cracked <1/8" width	Extensive cracking and hinge formation in ductile elements; limited cracking and/or splice failure in some nonductile columns; severe damage in short columns
Steel Moment Frames	Negligible	Minor local yielding at a few places; no observable fractures; minor buckling or observable permanent distortion of members	Hinges form; local buckling of some beam elements; severe joint distortion; isolated connection failures; a few elements may experience fracture	Extensive distortion of beams and column panels; many fractures at connections

TABLE 4 VISION 2000 SHEAR WALL AND FOUNDATION DAMAGE BY PERFORMANCE LEVEL (SEAOC VISION 2000)

	Performance Level			
Elements	**Fully Operational**	**Operational**	**Life Safety**	**Near Collapse**
Concrete Shear Walls	Negligible	Minor hairline cracking (0.02") of walls; coupling beams experience cracking <1/8" width	Some boundary elements distress including limited bar buckling; some sliding at joints; damage around openings; some crushing and flexural cracking; coupling beams-extensive shear and flexural cracks; some crushing, but concrete generally remains in place	Major flexural and shear cracks and voids; sliding at joints; extensive crushing and buckling of rebar; failure around openings; severe boundary element damage; coupling beams shattered, virtually disintegrated
Reinforced Masonry Walls	Negligible	Minor cracking (<1/8"); no out-of-plane offsets	Extensive cracking (<1/4") distributed throughout wall; some isolated crushing	Crushing; extensive cracking; damage around openings and at corners; some fallen units
Foundations	Negligible	Minor settlement and negligible tilting	Total settlements <6" and differential settlements <1/2" in 30 feet	Major settlements and tilting

TABLE 5 VISION 2000 BUILDING CONTENT DAMAGE BY PERFORMANCE LEVEL (SEAOC, 1996)

	Performance Level			
Element	**Fully Functional**	**Operational**	**Life Safety**	**Near Collapse**
Furniture	Negligible effects	Minor damage; some sliding and overturning	Extensive damage from sliding; overturning, leaks, falling debris, etc.	Extensive damage from sliding; overturning, leaks, falling debris, etc.
Office Equipment	Negligible effects	Minor damage; some sliding and overturning	Extensive damage from sliding; overturning, leaks, falling debris, etc.	Extensive damage from sliding; overturning, leaks, falling debris, etc.
Bookshelves	Negligible damage	Minor damage; some overturning and spilling	Extensive damage from leaks, falling debris, overturning, etc.	Extensive damage from leaks, falling debris, overturning, etc.

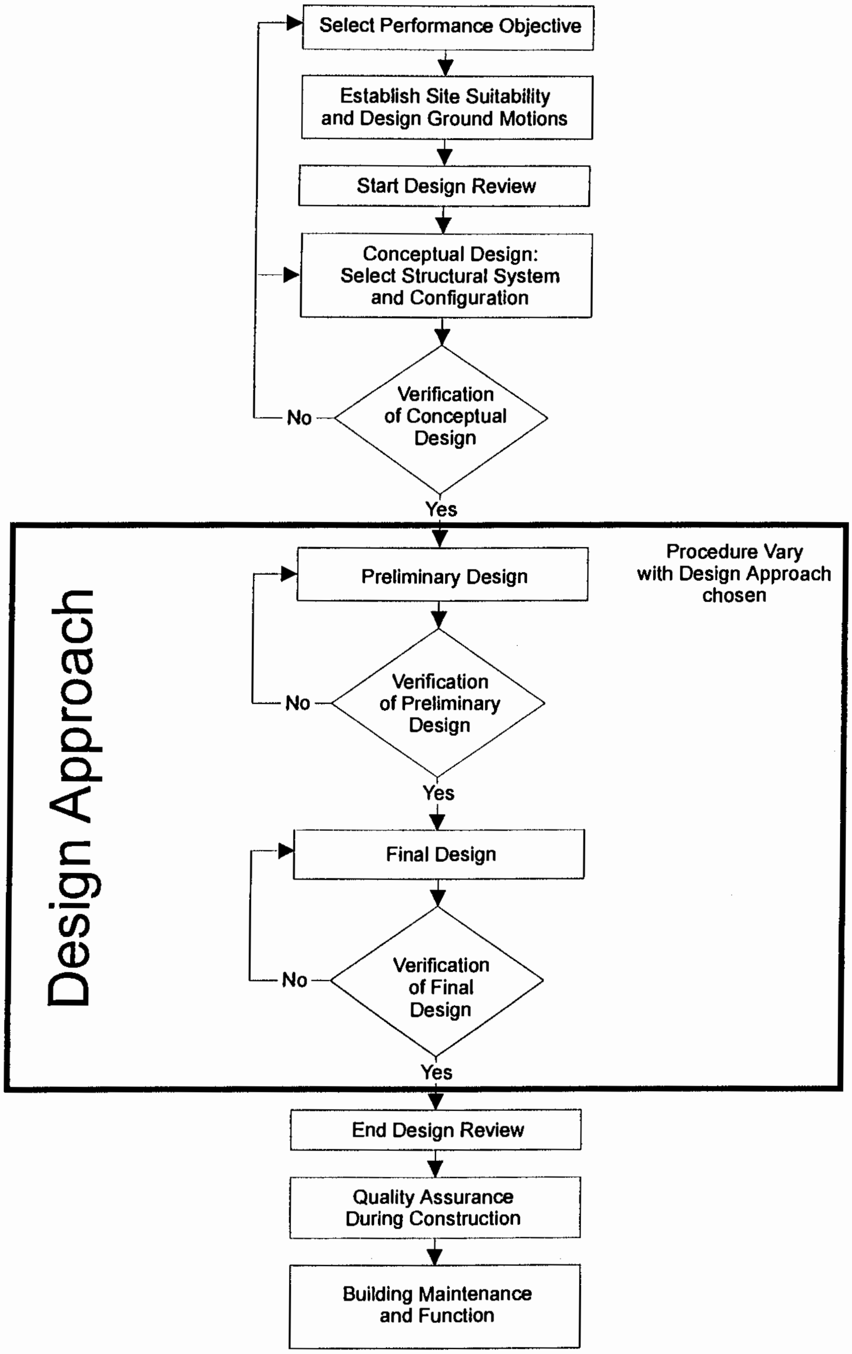

Figure 1: Vision 2000 methodology for performance-based engineering (SEAOC, 1996)

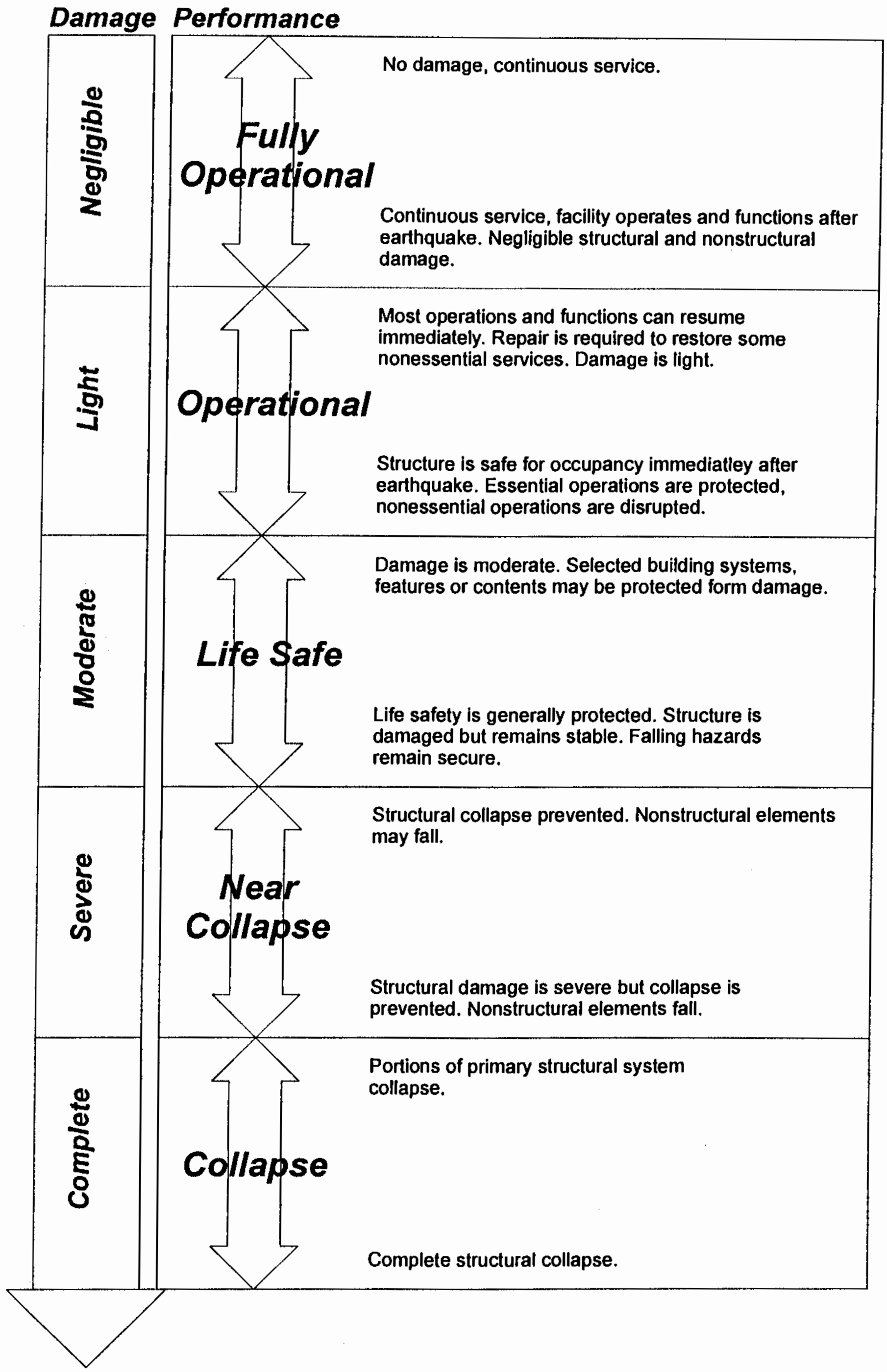

Figure 2: Vision 2000 spectrum of seismic damage states (SEAOC, 1996)

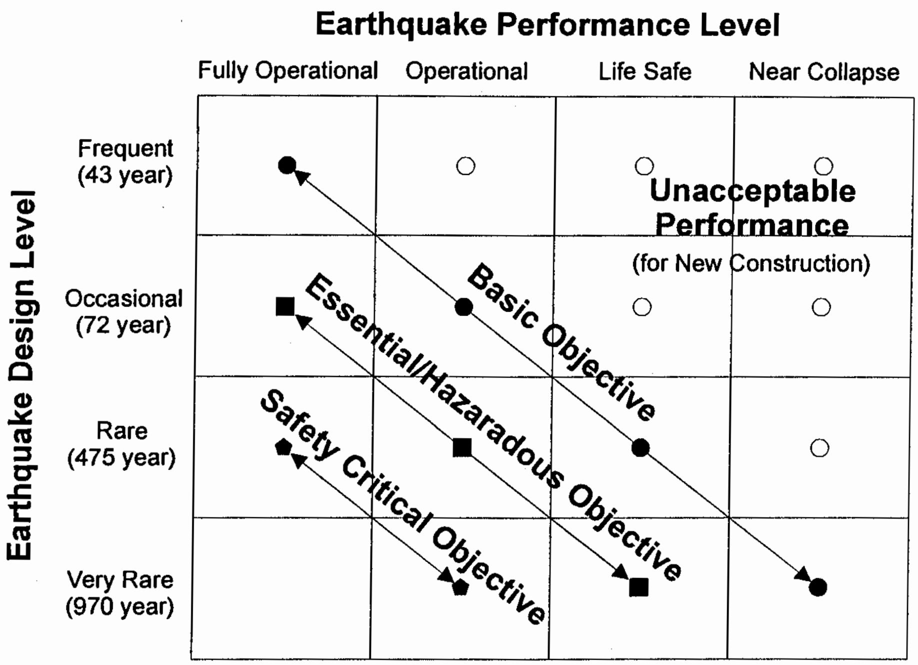

Figure 3: Vision 2000 recommended seismic performance objectives for buildings (SEAOC, 1996)