A HALF CENTURY OF PROGRESS IN THE SEISMIC EVALUATION AND STRENGTHENING OF EXISTING BUILDINGS

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Abstract

The evaluation and strengthening of existing concrete building has been an area of earthquake engineering that has evolved with great accomplishments since the earthquakes in Chile in the 1960s. The two authors have careers that span over 50 years since they first began their professional relationship and therefore they can provide examples of major accomplishments in ground motion estimation at a specific building site and also the analysis of the building performance for this ground motion. The first author was the PhD adviser of the second author in the early 1970's. Both authors' careers include being professors who worked with their students to perform research. Their careers also include participation in actual evaluation and strengthening projects of existing buildings. This paper presents some of the progress that has evolved over the last half century in earthquake ground motion estimation and the two areas of structural analysis: Structural Mechanics Structural Analysis and Structural Reliability Structural Analysis.

Keywords: Existing building evaluation and strengthening; Performance based design; Technology transfer; Structural reliability; Nonlinear dynamic analysis; Structural component testing; Structural system testing; Virtual testing; Limit states

1. Introduction

One can rightfully ask the question: If you are going to address progress in the last half century, what was the state-of-the-art just prior to that point in time? This is hard to define without omitting pioneer contributions dating back thousands of years. This is especially true prior to the expansion of rational structural mechanics' analysis methods in the early twentieth century [1], [2], [3]. Therefore, the authors, for the purpose of this paper, have started the story to be when Dr. Hart went to Chile to study the performance of concrete buildings in the early 1970's and met / worked with the chairman of this conference, Professor Rodolfo Saragoni, as part of a University of California / University of Chile cooperative research program.

Big progress in the last half century is the education of large numbers of doctoral and master's degree holders that have had at a minimum a good foundation in structural dynamics and earthquake engineering. One example of this is Professor Saragoni where Professor Hart was his PhD advisor at the University of California at Los Angeles. If one looks at the program for this conference, and the preceding conferences, then this progress is very evident and worldwide. Some universities require that doctoral dissertation research be published in a blind peer review journal and this is a major contribution to the advancement of the state of knowledge or what is often referred to as Technology Transfer. With the great advances of the past and certain advances in the future, the net benefit of this is that it becomes easier to communicate and implement new global derived technology in this new era of education and project specific earthquake evaluation and strengthening of an existing building.

Another big and fundamental change in the way the research community, structural engineers and decision makers view earthquakes and associated required decisions regarding the type and extent of seismic retrofit. In the 1960's, with few exceptions buildings were modeled as a two dimensional structural system and the response was calculated for only a few recorded earthquake ground motion records. For example, one could say it was the "1940 NS El Centro Earthquake" ground motion record era. The "glasses" we now use to plan our work and view the results are from a probabilistic point of view. For example, we now often use an ensemble of earthquake ground motion time histories developed for our specific site that are conditioned upon a 2% probability of exceedance in a 50-year exposure time.

Because of the progress in earthquake engineering, a fundamental question we are now able to answer in the strengthening of an existing building is: What earthquake are we going to focus on for our performance based building design? The answer depends on the decision maker's time frame of concern or what we will call the Exposure Time. Historically, the Exposure Time used in developing building codes and standards for new buildings has been 50 years. But with the wide spread acceptance of Performance Based Design, other Exposure Times have become the focus of attention. For example, the performance based design procedure, developed by the Los Angeles Tall Buildings Structural Design Council, requires the structural design of a new building to address Exposure Times of 30 and 50 years. In the commercial environment, the decision maker may select an even shorter Exposure Period that corresponds to, for example, the decision maker's expected tenure as Chief Executive Officer, or the Board of Directors fiscal business plan period. Other examples are projects for a major California hospital group and also a southern California university selected a 10-year Exposure Time for the seismic evaluation of their building.

A second fundamental question that we are able to answer based on progress is: What is the performance of the building that you consider to be a failure of the design to achieve for an earthquake that occurs in the selected Exposure Time? The answer starts with a definition of states of performance, called Limit States, and also the acceptable level of confidence that the defined limit state will not occur. In the last half century, the pioneering work from around the world in the 1950's and 1960's has been extended with great success.

This paper addresses what can be viewed as the foundation for a four leaf clover knowledge tree and the tree above the foundation, e.g. see Fig. 1. One leaf discusses the role of the structural engineer of record and the perspective he or she uses to determine if structural plans and specifications are acceptable to give to the contractor for construction. Another leaf addresses the evolution of earthquake engineering in approximately the last half century of the development of earthquake ground motions for the different Exposure Times used in Performance Based Design. The third leaf addresses the evolution over this same time period of the contribution of Earthquake Engineering research to do structural analyses to perform a building design. Finally, the fourth leaf addresses the data base of laboratory physical test results and very sophisticated micro-model virtual test results that benefit the experience base for the specific limit states that define the performance of the building under evaluation.

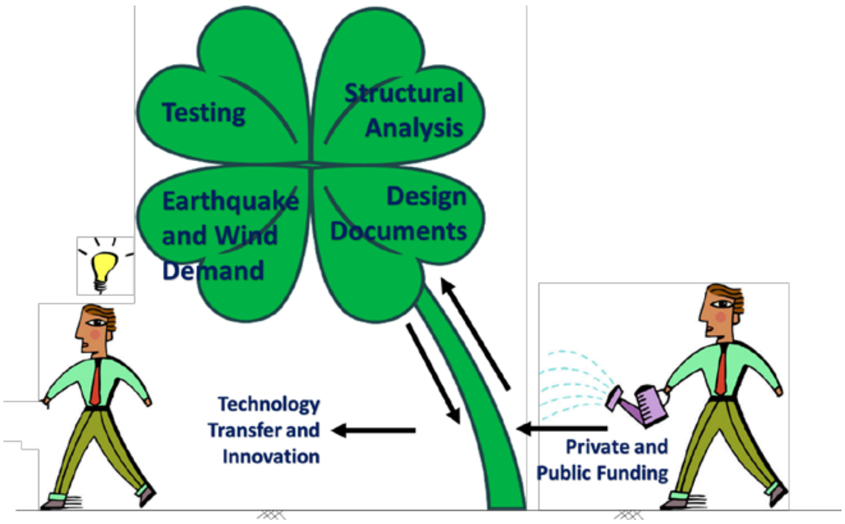

Fig. 1 The Structural Engineers Four Leaf Clover

2. The Structural Engineer of Record and Performance Based Design

Structural engineers always develop designs using their professional education and experience when viewed through what can be called Bayesian Structural Reliability "glasses". In the "olden" times, prior to structural mechanics' equations, structural experience ruled. Now we can use equations and combined experience and the results from analyses use of equations. Committees recommend equations with different levels of sophistication what we now call Structural Mechanics' Structural Analysis (SMSA) models. Advances in structural behavior of concrete, masonry, and other materials enable us to develop even more sophisticated SMSA models. We also now use equations in each Bayesian Structural Analysis that use our prior estimation of design concepts and / or numerical input to analysis models in light of project specific information as it relates to the target performance objective. The result is a revision of our Prior Design to obtain our Posterior Design. In the last half century, the area of structural engineering called Structural Reliability Structural Analysis (SRSA) has made and continues to make significant advances.

One excellent way to visualize the steps that the structural engineer should take, based on our last fifty years of research, prior to performing an evaluation and strengthening of an existing building is to view this journey using the four components of uncertainty identified in the ATC-63/FEMA P695 report [4]. The four parts are: (1) Earthquake Demand, (2) Testing, (3) Structural Mechanics Analysis, and (4) Design Documents, see Fig. 1. The next section of this paper provides insight into Part (1). Part (2) is a critical part of all projects and it has come in the last half century to involve two types of testing which are: Field Investigation and Laboratory Testing. The time and expense of both of these types is always a concern and requires a decision as to the level of confidence we wish to achieve that our building will perform as we intend it to. Table 1 is one quantification of confidence that has evolved and been accepted by many earthquake engineers. In the most basic description, we start with a prior testing data base and then supplement it with project specific testing.

Thus, in structural reliability language we have learned over the last half century to start with a Prior state of knowledge and hope to evolve it into a project specific Posterior state of knowledge.

Table 1 – Confidence Scale [5]

Confidence	Probability
Almost Certain	90 to 99.5% sure
Very Likely	75 to 90% sure
Likely	60 to 75% sure
Medium Chance	40 to 60% sure

Table 1 is one way to verbally communicate confidence and a very important other way is from ATC-63/FEMA P695 and it places confidence in four categories which are Superior, Good, Fair and Poor. The important point in the context of this paper is that we can, and now should, always receive a transparent assignment of confidence and the reasons for placement in that confidence category. In the early years of earthquake engineering, in the 1960's and 1970's which did not record talks and questions, we did not consider it offensive to require all information that formed the basis for an opinion. Now we can place confidence in one of these categories, but the open shared communication in, for example the question and answer session of conferences like this one, is very limited and so the group sharing via group discussion is not typically optimal.

To illustrate this relationship between Prior and Posterior and how it can be based on a scientific foundation, consider one structural reliability structural analysis approach which uses the following equation relating the prior and posterior estimates of the mean value of a structural parameter [6] and [7], which is

$$\bar{Y}_u = C_1 \bar{X}_t + C_2 \bar{Y}_p \tag{1}$$

where

$\bar{Y}_u =$ posterior estimate of parameter (e.g. mean value of maximum concrete compressive strength)

$\bar{Y}_p =$ prior estimate of parameter (e.g. pre-testing mean value of maximum concrete compressive strength)

$\bar{X}_t =$ estimate of parameter from *n* pieces of "test" data (i.e. sample mean value)

$$C_1 = \left[1/\left(1+\left(R/n\right)\right)\right] \tag{2}$$

$$C_2 = \left[1/\left(1+\left(n/R\right)\right)\right] \tag{3}$$

$$R = \left(\sigma_x / \sigma_{y_p}\right)^2 = \text{relative uncertainty ratio (prior to posterior)} \tag{4}$$

σ_x, σ_{y_p} = standard deviation of mean prior to, and after testing, respectively

n = number of pieces of information (e.g. concrete cores)

As structural engineers, we must study the assumptions upon which this Eq. (1) is based, assign values for $\bar{Y}_p$ based on our interpretation of information, and the result is only information upon which we evaluate if we are Almost Certain that our design meets our performance objectives. Further discussion of this role of prior and posterior estimations of structural parameters is summarized in the LATB 1, 2 and 3 reports [8, 9, 10].

Eq. (1) shows us that the revised, posterior estimate of a value of a parameter for a limit state that defines performance is the weighted summation of: (1) the result of our readings of journal papers, conference papers and reports combined with ours and others unpublished personal project test data values, and (2) the values obtained from a numerical value of new information which is typically from new tests. What we are now able to benefit from in a scientific way is the use of the Prior often not up to current testing protocols combined with new testing within a project specific financial budget to obtain the Posterior values for parameters that define limit states.

3. The Earthquake Ground Motion

The development of earthquake ground motions for a specific building site can be viewed as one of two options. One option is to develop "synthetic" time histories and the other option is to develop "modified time histories". Over the last half century, we have obtained strong motion earthquake records, and then expanded our database of information, especially in the last decade so that large earthquakes are recorded simultaneously by hundreds of instruments. Despite this improvement, today's world has a scarcity of records for very large magnitude earthquakes [11] or records obtained in rock impel to develop such records in "synthetic" way on by "modifying time histories".

The good behavior of mid-rise buildings during the 1906 San Francisco earthquake designed only to wind forces by Chicago structural engineers with an equivalent shear base force of only 0.02 of the weight of the building is responsible of the beginning of requirements of strengthening of existing buildings in California. Our focus then started on the total earthquake and placed a very visible desire for us to obtain real recorded earthquake ground motion records. Other illustrations of this focus and experience are the shear base force of 0.02 of the building weight which was comparatively smaller than 0.09 proposed by Panetti [1] after the 1908 Messina, Italy earthquake or the 0.10 to 0.20 proposed by Naito and Sano [2], [3], and verified its good performance for reinforced concrete buildings in the 1923 Kanto, Tokyo earthquake in Japan.

The development of the strong motion accelerograph in USA. by John Freeman at the USGS in 1930 allowed us to obtain the first accelerograms which began after the "1940 NS El Centro Earthquake" response spectra. Today we know that El Centro 1940 NS record corresponds only to a small quake record [21], [22].

The recorded accelerograms for the 1994 Northridge earthquake near the fault forced to introduce near source factors to amplify the design response spectra in the UBC '97 [12]. These near source factors in the first miles duplicate former spectra making it necessary to strengthen designed buildings with former codes. These results showed the deep impact of recorded earthquake ground motion in the structural design.

In the context of the previous section, we study these records as they might relate to a specific building site and always use professional judgment to develop a site specific estimation of ground motion on deterministic (DSHA) or probabilistic (PSHA) bases. The authors of this paper prefer in most cases to develop the earthquake ground motion at a site using the most recent research results in the literature and an ensemble of synthetic time histories. [13] The reasons for this is that scaling records of small earthquakes to simulate records of large earthquakes is like the scaling of "a cat to be a lion", it will never reproduce its main characteristic which is dangerous. The evolution of energy with time of a large earthquake is completely different than small earthquakes and this is particularly critical for dynamical nonlinear analysis of structures. In particular records of small quakes are poor in longer period content than records of large magnitude earthquakes. This is a problem for instance in the method proposed by Irikura et al. [20] based in Green's functions. This situation is particularly critical for performance design analysis of tall buildings.

The pioneer Freeman and USGS contribution allows today to have hundreds of records of recent large earthquakes from high dense array in common time. In Fig.2 we can see the large difference in amplitude, frequency content, duration, evolution of energy due to asperities effects for accelerograms of the mega-earthquake Chile, El Maule 2010, Mw=8.8. Earthquake ground motion records can be viewed with the benefit of earthquake engineering contribution ones the last half century.

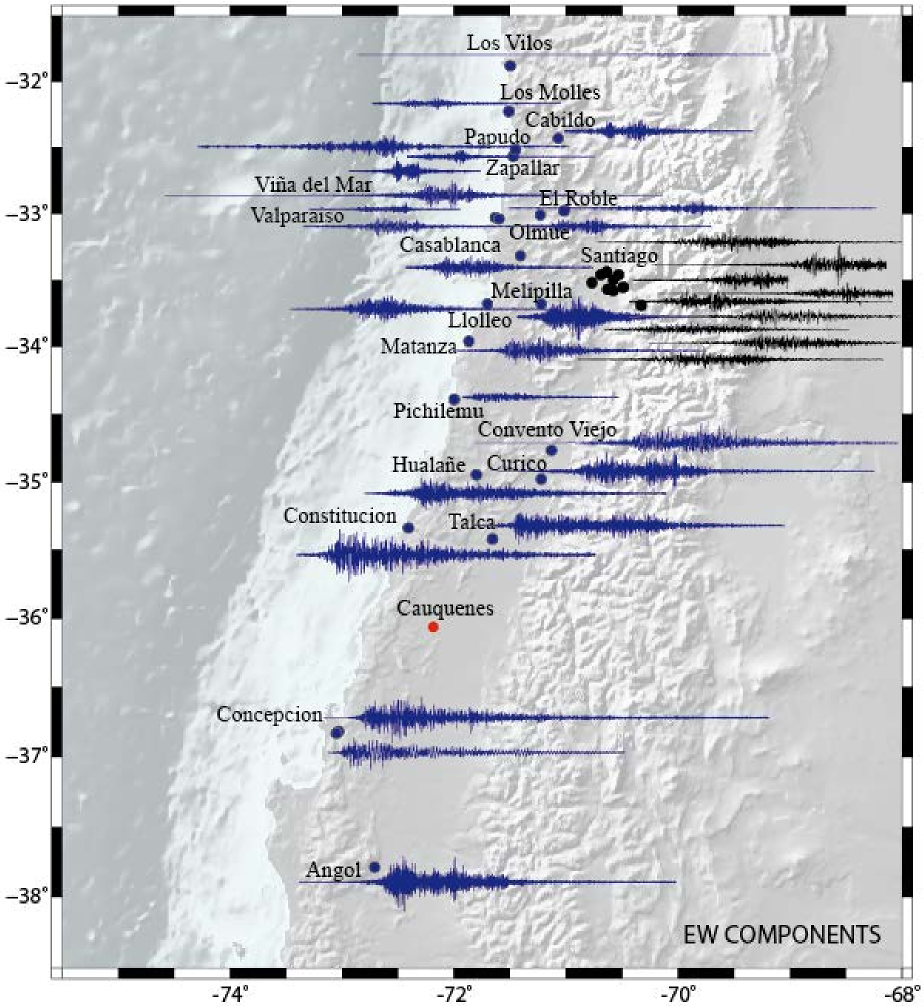

Fig.2 Accelerograms Mega- earthquake Chile 2010, El Maule Mw=8.8 showing the differences in amplitude, duration and frequency content [11]

The evolution of earthquake energy with time in records is controlled by the type of seismic waves arriving from the source and modified by soil response [14], [15].

Others prefer to modify time histories to emulate earthquakes records estimated to happen during different exposures of time considered in the Performance Based Design.

The selected time histories for the design in both cases of "synthetic" or "modified time histories" are required by codes to satisfy certain constraints referred to as their mean response spectra compared with standard design spectra. This elastic benchmarking is a limitation for the full development of nonlinear dynamic analysis [NDA] in performance design should be analyzed in future.

In Chilean practice for nonlinear analysis of Performance Based Design [16], [21], [22] the potential destructiveness factor is used since it forecasts better the global ductility of building.

4. The Structural Analysis: Part I, The Structural Mechanics' Structural Analysis

Structural Analysis can be viewed with the benefit of earthquake engineering contributions over the last half century as having two parts. One part is the performance of a structural mechanics' structural analysis model that can be as simple as a static analysis or as complex as a PhD dissertation using the latest theories of micro-level fracture mechanics. The other part is the utilization of the theory of structural reliability to calculate whether a demand on a limit state exceeds its capacity, and if not, whether the demand is sufficiently less than the capacity to provide an acceptable level of safety against the failure of a limit state. The first part is called the Structural Mechanics' Structural Analysis (SMSA), and the second part is called the Structural Reliability Structural Analysis (SRSA).

The SMSA options are used to estimate the performance of an existing building because of both theoretical and computer advances in the last half century and many are typically area and building specific. However, education and experience in the last half century has taught us that we must do multiple levels of SMSA prior to submitting our construction plans for approval to the building official to obtain a construction permit. For example, Dr. Robert Englekirk always did a mental SMSA of a proposed structural system sometimes without even his famous envelope of notes he provided to this project managers or internal peer reviewers. Another example of SMSA is Mr. John Abruzzo doing his "hand" analysis with or without his computer spreadsheets. At a higher level of confidence are Nonlinear Dynamic Analyses (NDA) as performed by Weidlinger Associates Inc. for the 9/11 New York World Trade Center Collapse or a SRSA using such computer programs as PERFORM3D including stiffness and strength degradation. Professor Tom Caughey, a very analytical structures and mechanics engineer, taught the lead author a very important lesson and it was "do not go to the computer until you think you know the answer".

We have learned in the last half century that a NDA is the performance of an existing building's structural system that must at a minimum use the structural engineer's best estimates of the Expected Value of a required structural analysis input parameters for the selected structural analysis computer program. These Expected Values can be the Mean Value or the Median Value. For most parameters that are requiring this Expected Value, there is little difference between the Mean and Median because the Coefficient of Variation of the input parameter (i.e. the random variable) is less than 35%.

With the ever increasing speed of our computers to perform the computations in our structural analyses, we derive ever increasing benefits to uncertainty reduction and increased confidence. We are also benefiting from the open world market in structural analysis software. What unfortunately is still less than desirable is the reluctance of many earthquake/structural engineers to utilize this benefit because of many reasons including: (1) lack of desire to do a better structural analysis model; (2) not wanting to have an external Peer Reviewer look over their selection of the particular software or, more commonly, their selection of values for computer input parameters for the specific building under evaluation and strengthening; and (3) perhaps the most important is the extra pressure that the benefits derived from this technology transfer are hard to read in the paper trail of the technology transfer placed on the building official because what is being used is an equation not in the prescriptive building code provisions.

A NDA that uses time histories of ground motion requires a minimum of seven to eleven or more time histories of ground motion. These time histories are typically an ensemble of earthquake records conditioned upon the occurrence of a Maximum Considered Earthquake (MCE). The NDA has been shown to be even more informative and powerful if the ensemble of records corresponds to multiple levels of earthquake demands as is done in an Incremental Nonlinear Dynamic Analysis (INDA).

The evolution in the last half century of SMSA and design has resulted in the general acceptance of Performance Based Design. With this comes the need to define limit states that define the performance objective in words and numbers. But even prior to this entering our codes, the term "Performance Based Design" was the basic goal of a good structural engineer which was to try to control with the design certain important performance objectives. For example, in the 1970's a topic of focus was the torsional response of buildings. Another was the importance of trying to estimate when the "elastic" response of the structural component or system ends and where it occurs first. Structural analysis has taught us that, as we continue to develop more sophisticated structural analysis computer programs to address building performance, we continue to combine our past education and experience with the results of our structural analyses to obtain a final design in which we have an estimated level of confidence that meets our performance objectives.

The most common performance objectives that our structural analysis seeks to provide input for the final structural design are what we in structural reliability structural analysis call Conditional Structural Analysis. The analysis focuses on estimating the performance of our building for the occurrence of a "Service Level" and an "Ultimate Level" earthquake. The structural analysis model for tall buildings for the "Service Level" earthquake is most commonly called a Linear Analysis Structural Model. Fig. 3 [17] shows the experimental force versus deflection curve for a structural component. Starting about half a century ago, Professor Mete Sozen from the University of Illinois at Urbana-Illinois, and other earthquake engineers around the world, suggested the phrase "Effective Stiffness" and our linear structural models used structural parameters to define stiffness that resulted from drawing a line starting at the origin of the load deflection curve to the point on the curve corresponding to a selected value of displacement. This approach is basically the same as many engineers today call Displacement Based Design. What evolved over the years started with Elastic Plastic (up and then horizontal), Bi-Linear (one up slope followed by another up slope), and then the Multi-Linear sloped curve. The Sozen Effective Stiffness approach is still an option with value as part of a linear structural analysis model because the displacement of interest can transition from small to large displacements and obtain linear analysis results. In the context of this paper is that over the last half century, we have an ever increasing position of support for the requirement to define the "Elastic" or "Yield" limit states and perform a Linear Structural Analysis to help develop our design.

The story of structural analysis usually continues with what is called the development of nonlinear structural analysis models. These models started about a half century ago and now have advanced both in the research community and structural computer software design community that markets its computer codes. The fundamental point that exists is that in all cases the more money that is spent on preparing a nonlinear model and then studying / learning from the results, the more confidence we have in our estimation of structural performance and the demand on our limit states. For example, we have evolved from simple extensions of linear models without cyclic strength deformation to today's commercially available codes where we can model strength degradation. If we believe it is sufficient, we also provide the necessary nonlinear analysis results for finalizing our design representing the structural system or one of its components (e.g. a shear wall) with a single degree of freedom nonlinear analysis model. So we have seen nonlinear structural analysis models that can be developed and results studied in terms of days up to others that require many months.

An acceptable question is how do we select the "best" nonlinear structural analysis model for our building. The answer is simple and it is the one that is acceptable to the Structural Engineer of Record and then is approved by a Peer Review panel of experts in testing, analysis and design. Nothing has really changed over the last half century in this regard because great structural engineers such as Robert Englekirk, Roy Johnston, Henry Degenkolb, and others always did this with either internal staff members or outside consultants serving as Peer Reviewers.

5. The Structural Analysis: Part II, the Structural Reliability Structural Analysis

When Dr. Hart was obtaining his Master's Degree at Stanford in 1965 Professor Jack Benjamin introduced the class to the area of probabilistic structural dynamics and also Bayesian Decision Making. He, with others around the world who were structural engineers by education and experience, recognized the potential benefits to educating students, writing journal papers and integrating in a very scientific way the mathematics of probability theory into structural, earthquake and wind engineering.

We have always recognized that for a given design life, e.g. 50 years, the maximum earthquake ground motion in that time and also the capacity of a structural member on a structural system are random variables. We select the design demand to be greater than the expected demand, and the design capacity to be less than the expected capacity. This is where the Structural Reliability Structural Analysis has evolved and helped us be better structural earthquake engineers.

If an ensemble of "*n*" earthquake ground motions are provided, then we have "*n*" values of demand on each limit state. Figs. 3 and 4 [17] shows the definition of eight limit states that define performance. The numbers in squares in Figs. 4 and 5 show the response obtained from an expected value structural analysis. If the number in the square is to the left, e.g. limit state 3 in the square to the left of the number 3, then the response satisfies the limit state and failure has not occurred. However, since there are many ground motion time histories in our ground motion ensemble, e.g. "*n*", we can then determine what percentage of them produce a response where the numbered square three is to the right of 3 and thus failure. This is one approach used in structural reliability analysis, but another is to calculate the mean (i.e. average or expected) value of the deformation that occurs for the 3 in the square and to set a design safety margin, denoted 3* in Fig. 5 [17] that is based on the coefficient of variation of the "*n*" response results and a Safety (or Reliability) Index that reflects the negative consequences of the performance not meeting the limit state.

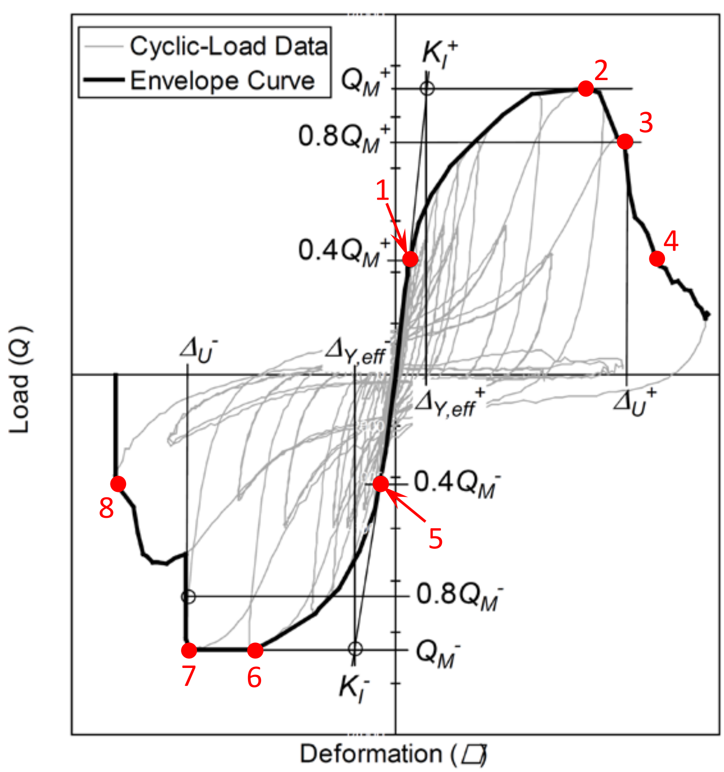

Fig. 3 Performance Limits for Structural Member
(derived from ATC 63-1/FEMA P795, 2011, p. 2-12)

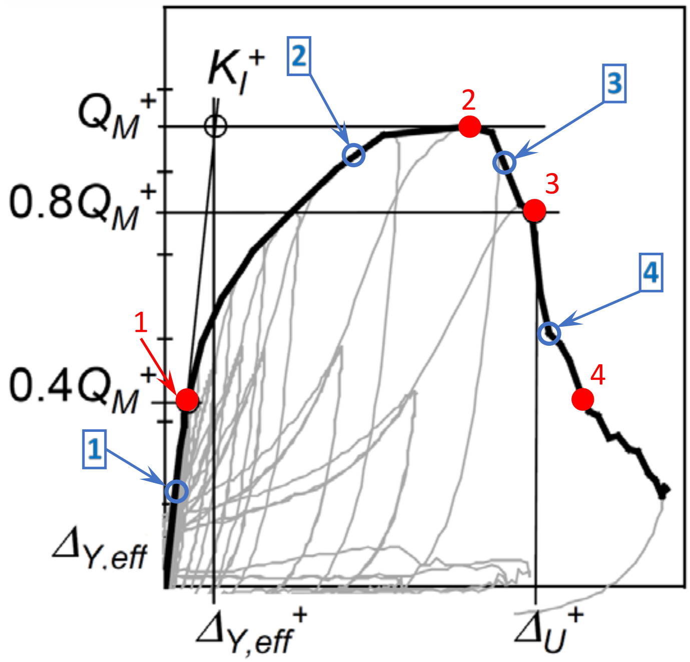

Fig. 4 Design Limit States for Structural Members (Flags in PERFORM 3D Model)
(derived from ATC 63-1/FEMA P795, 2011, p. 2-12)

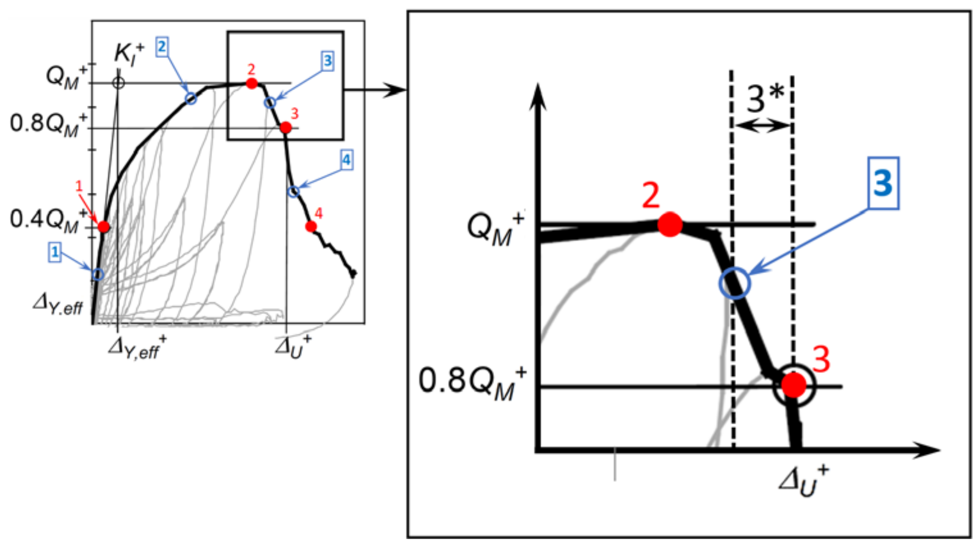

Fig. 5 Expanded View of Descending Branch of Backbone, Between Points 2 and 3
(derived from ATC 63-1/FEMA P795, 2011, p. 2-12)

It is good to see the percentage of university and college structural engineering education programs that offer courses in the area of SRSA increase over the last half century. However, it is not required as part of the accreditation program and also the classroom hours devoted to SRSA is way out of balance when compared with the other part of a structural analysis: Structural Mechanics Structural Analysis (SMSA). A positive note is that more than ever structural/earthquake engineering demands continue career education and with the electronic era, the real cost of publishing books has decreased and is more affordable in structural reliability has seen many excellent books. Also, of special note is the publication of ATC-63 funded by FEMA has broadened the audience for reading in structural reliability.

6. Conclusions and Summary

The century of progress has been great for the individuals who were able to use their special gifts to advance technology, and for the public who are able to have more confidence in not only effectively communicating their performance objectives, but also as earthquake engineers we are now able to perform analyses so that the performance of buildings is acceptable with a quantifiable level of confidence. The reader is referred to two recently accepted papers to the Wiley journal "The Structural Design of Tall and Special Buildings" [18,19] as well as the LATB reports mentioned earlier to expand upon many of the ideas presented in this paper.

The passage of this half century has seen the transfer of learning via prescriptive provisions and commentary in the building code. This transfer has resulted in a lengthy code and many prescriptive provisions that merit entry for the "typical" structural engineer who designs low rise buildings and non-trophy residences. It also has started to produce commercial computer software that combines with the total scope of services in building design a Mega Computer Software that will diminish both the time and the knowledge of real building expected earthquake records required by the structural engineer of record. But what we have experienced is the start of an era called Performance Based Design that when based on a transparent foundation of Structural Reliability rewards uncertainty reduction that incorporates the results of earthquake engineering efforts in the last half century and also the creative and scientifically-based future learning. Therefore, it is the author's opinions that we are entering a golden period of earthquake/structural engineering. When viewed in the context of the four leaf clover tree in Fig. 1, we are entering an era of Beautiful Earthquake/Structural engineering and we say thank you to the gardeners who planted this tree and we must be the gardeners who care for this tree.

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