Structural Analysis for Performance-Based Earthquake Engineering

- Basic modeling concepts
- Nonlinear static pushover analysis
- Nonlinear dynamic response history analysis
- Incremental nonlinear dynamic analysis
- Probabilistic approaches

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The next set of slides deals with performing nonlinear RESPONSE HISTORY analysis. Note the emphasis on the word RESPONSE. In the past, the terminology TIME HISTORY analysis has been used. Upon some reflection it is clear that this makes little sense because it is not the history of time that is of interest… it is the history of the response of the structures that we are interested in.

Nonlinear Dynamic Response History Analysis

Principal Advantage: **All** problems with pushover analysis are eliminated. However, new problems may arise.

Main Concerns in Nonlinear Dynamic Analysis:

1) Modeling of hysteretic behavior
2) Modeling inherent damping
3) Selection and scaling of ground motions
4) Interpretation of results
5) Results may be very sensitive to seemingly minor perturbations

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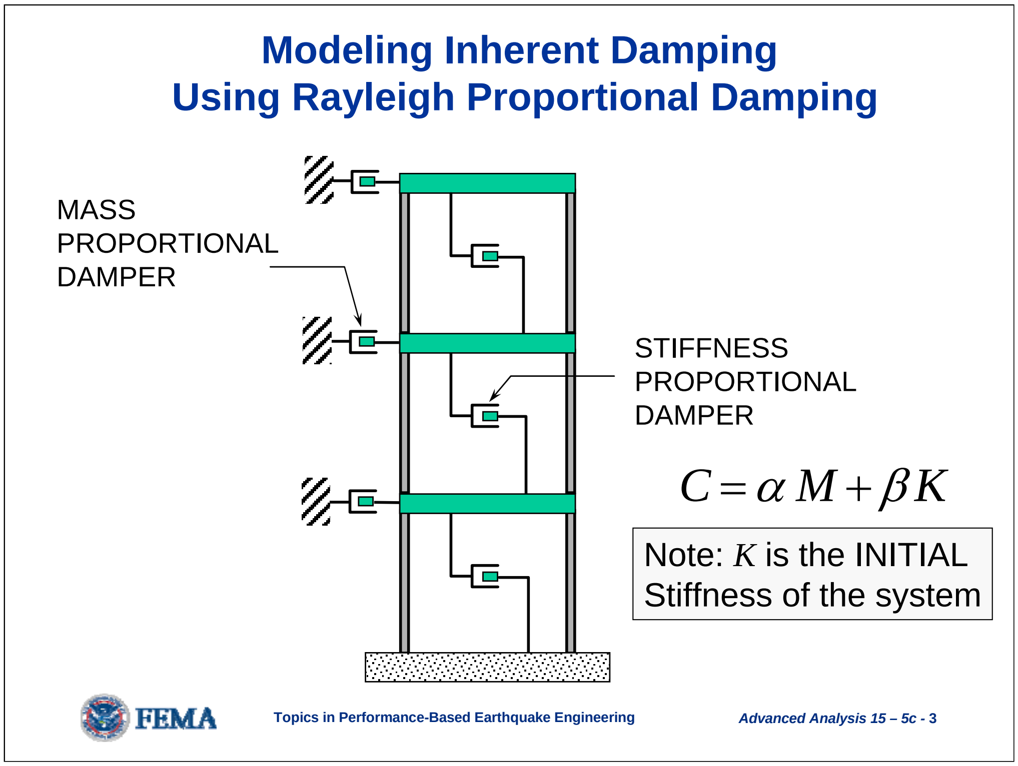
Response history analysis has the strong advantage of eliminating all of the problems associated with pushover analysis. Unfortunately, a new set of problems arises, some of which are listed here. Due to the fact that some of these problems may be insurmountable in the framework of a deterministic analysis, a probabilistic framework is being developed. The probabilistic approach is described very briefly at the end of this topic.



In DRAIN (and most other nonlinear dynamic analysis programs) the inherent damping is represented as Rayleigh damping, which produces a damping matrix that is proportional to mass and stiffness. Such a damping matrix will be diagonalized by the mode shapes, allowing for full decoupling of the equations of motion. Ironically, this decoupling is not utilized in DRAIN because the full set of coupled equations are solved step by step in the time domain.

The slide shows a physical interpretation for Rayleigh Damping. Note that base shear will be lost through the mass proportional dampers, which may be referred to a “viscous sky hooks”.

Full Rayleigh damping, wherein alpha and beta are specified globally, should be used only to represent low amounts of inherent damping (say < 5% critical). As explained later, even this should be done using extreme caution as unintended effects easily destroy the accuracy of the analysis.

In DRAIN, the stiffness proportional damping factor (beta) may be set on an element by element basis. This makes it possible to represent discrete dampers as shown later.

It is also important to note that the stiffness proportional part of Rayleigh damping (as implemented in DRAIN) is proportional to the INITIAL stiffness of the system. As explained later, this can lead to significant problems if the analyst is not careful.

Rayleigh Proportional Damping

Select Damping value in two modes, ξ_k and ξ_n

Compute Coefficients α and β:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2\frac{\omega_k \omega_n}{\omega_n^2 - \omega_k^2} \begin{bmatrix} \omega_n & -\omega_k \\ -1/\omega_n & 1/\omega_k \end{bmatrix} \begin{Bmatrix} \xi_k \\ \xi_n \end{Bmatrix}$$

Form Damping Matrix $C = \alpha M + \beta K$

FEMA Topics in Performance-Based Earthquake Engineering Advanced Analysis 15 – 5c - 4

When modeling system-wide inherent damping the analyst specifies damping ratios in any two modes. Given the modes' frequencies the proportionality factors alpha and beta may be determined.

Rayleigh Proportional Damping (Example)

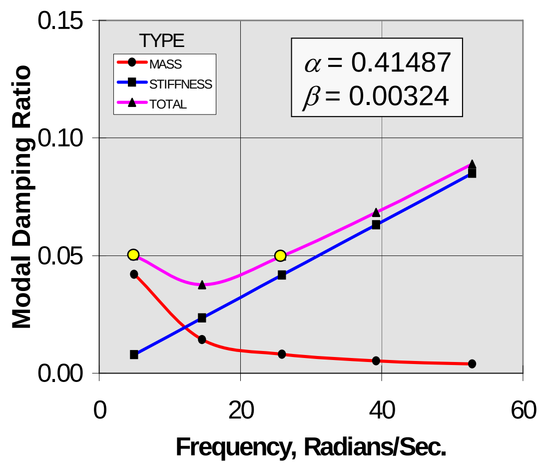
5% Critical in Modes 1 and 3

Damping in any other Mode m:

$$\xi_m = 0.5 \begin{bmatrix} \frac{1}{\omega_m} & \omega_m \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix}$$

Structural Frequencies

Mode	ω
1	**4.94**
2	14.6
3	**25.9**
4	39.2
5	52.8



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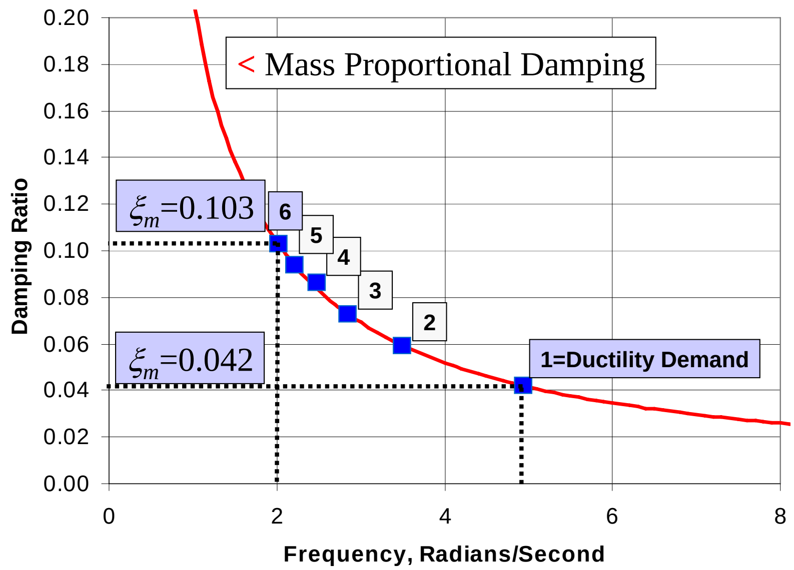
Advanced Analysis 15 – 5c - 5

An example of a 5-DOF system is shown here. Damping has been set as 5% critical in modes 1 and 3. Damping in the other modes is determined from the formula shown in the upper right. In this case the damping in the second mode is less than 5%, and damping in modes 4 and 5 is greater than 5%.

Note that the stiffness proportional component of damping increases with frequency and the mass proportional component decreases with frequency. For MDOF systems, Rayleigh damping has the tendency to suppress the higher modes. This is a good thing if these modes do not contribute to the response. It can be a bad thing if these modes are important, such as potential resonance in higher modes due to, say, vertical ground accelerations.

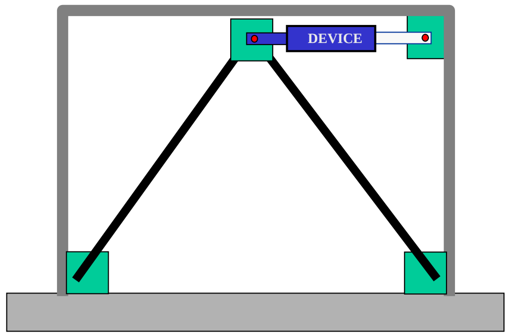
Loss of stiffness, frequency shift, and higher mass proportional damping



FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 6*

One of the potential problems with Rayleigh damping is the fact that the effective damping ratio for the various modes may increase as the structure softens and the frequencies shift to the left (as indicated on the plot). This can happen for bilinear systems without degrading stiffness if the system is under sustained yielding. The higher the ductility demand, the greater the apparent increase in damping. It is recommended, therefore, that the mass and stiffness proportional damping constants alpha and beta be based on frequencies consistent with a reasonable ductility demand.

Modeling Linear Viscous Dampers in DRAIN



Note: Nonlinear Damping is NOT Available in DRAIN.

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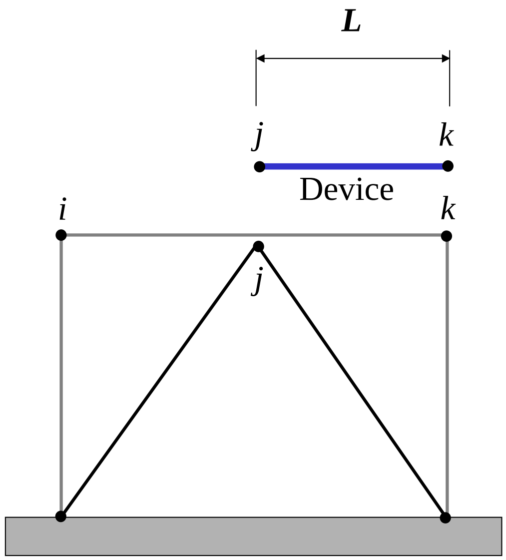
Discrete damping, such as applied by a viscous fluid damper, may be easily modeled in DRAIN. The only limitation is that the damping must be linear, e.g. the damping fore is directly proportional to the deformational velocity in the device. If it is important to model nonlinear viscous dampers one must use SAP2000, RAM Perform, or OpenSees. It should be noted that nonlinear dampers are almost always preferred because of their capability to “yield” under large deformational velocities.

Modeling Linear Viscous Dampers in DRAIN



Use element stiffness proportional damping.

$$K_{Damper} = \frac{AE}{L}$$

$$C_{Damper} = \beta K_{Damper}$$

For low damper stiffness:
Set $A=L$, $E=0.01$

use $\beta = C_{Damper}/0.01$

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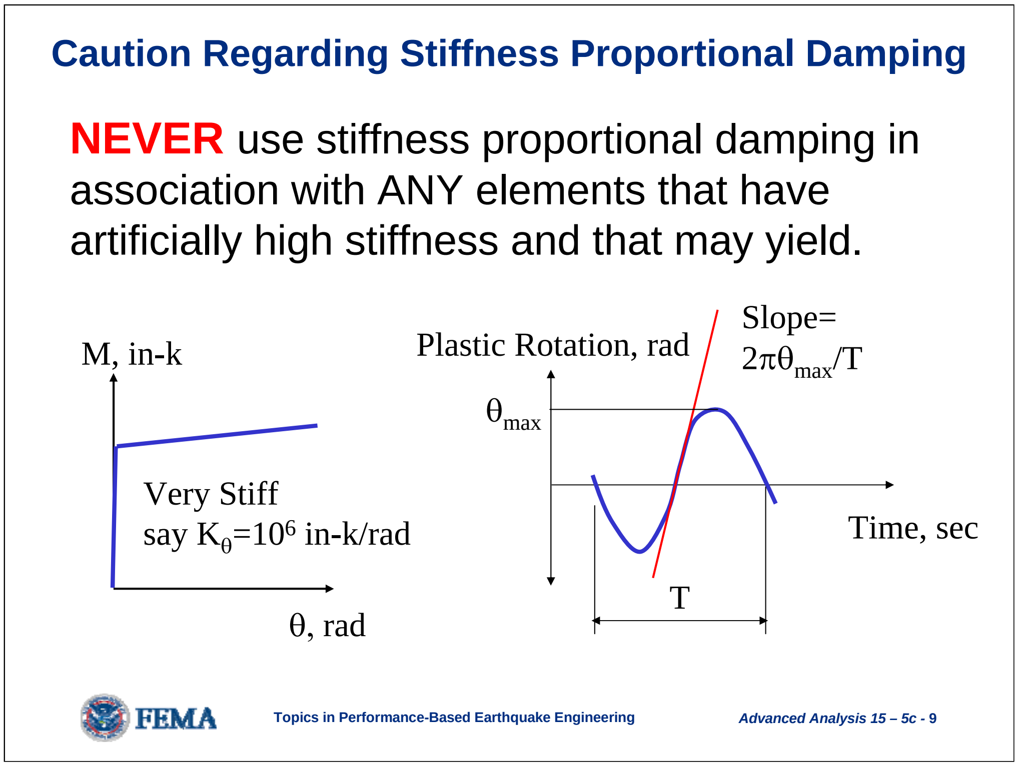
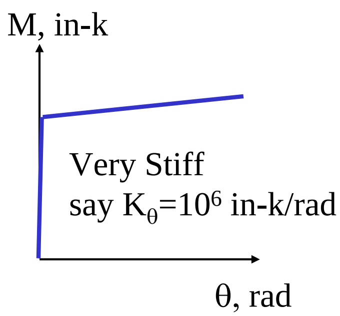
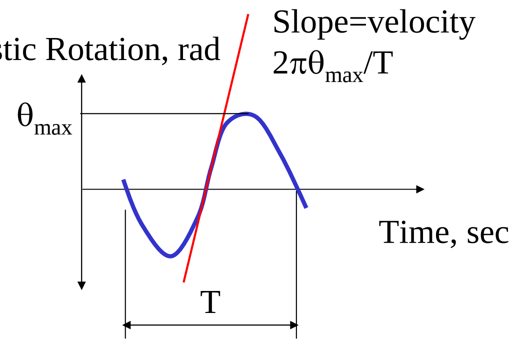
This slide illustrates the technique used to model a linear viscous damper in DRAIN. The basic idea is to use a Type-1 truss bar with a very low stiffness and with a very high Beta value. The product of the stiffness and the Beta value should be equal to the desired damping coefficient, C. The use of a very low stiffness is consistent with the behavior of a viscous fluid damper which has a near zero storage stiffness (if excited below its cutoff frequency). If it is required to model a viscoelastic damper this can be done by appropriate selection of the properties.

It must be noted that the flexibility of the brace may have a profound effect on damper effectiveness. Full effectiveness will be achieved with a very stiff brace. Near zero effectiveness will be achieved with a very flexible brace. The analyst should perform sufficient analysis to determine the effect of the actual brace stiffness on the effectiveness of the device.



This slide shows how dangerous it can be to arbitrarily assign stiffness proportional damping to a structure. Here, a plastic hinge is modeled with an initially high rotational stiffness as is common. The force-deformation curve for the hinge is shown at the left. The diagram on the right is a response history of the plastic rotation in the hinge while the hinge is yielding. The slope of the red line indicates the maximum deformational velocity in the hinge. This velocity can be given as shown if it is assumed that the response is harmonic over the time *T*.





Viscous Moment in Hinge = $K_\theta \beta (2\pi\theta_{max}/T)$

Assume θ_{max} = .03 rad, T=1.0 sec, β=0.004

$M = 10^6(0.004)(2\pi(.03)/1.0)) = 7540$ in-k

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Here, some reasonable numbers are given. Say the maximum rotation is 0.03 radians, the period *T* is 1.0 seconds, the initial stiffness is 10^6 in-k/radian, and the stiffness proportional damping factor beta is 0.004. The viscous moment in the hinge will be 7540 inch-kips. This is a completely fictitious but very significant moment which will be added to the plastic moment in the hinge to determine the actual moment. It is likely that dozens of response history analyses have been run with this effect which remained undetected by the analyst.

NEHRP Ground Motion Selection

- Ground motions must have magnitude, fault mechanism, and fault distance consistent with the site and must be representative of the maximum considered ground motion
- Where the required number of motions are not available simulated motions (or modified motions) may be used

(Parenthesis by F. Charney)

How many records should be used?
Where does one get the records?
How can the records be modified to match site conditions?

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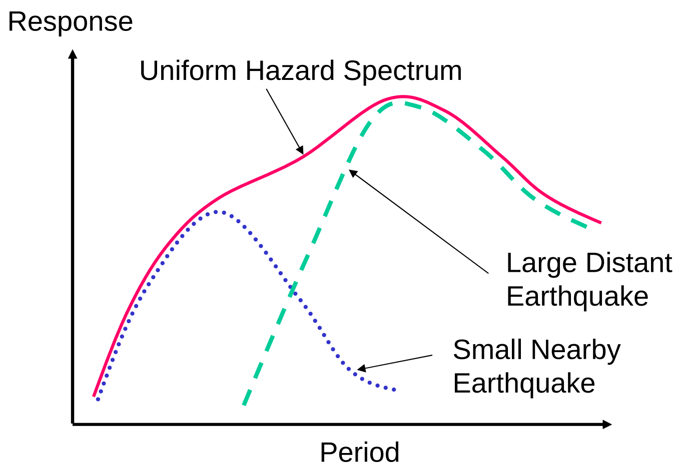
One of the most difficult aspects of response history analysis is the selection and scaling of ground motions. This slide asks some of the relevant questions, many of which will be addressed in the next several slides.

Use of Simulated Ground Motions

Simulated records should **NOT** be used if they have been created on the basis of spectrum matching where the target spectrum is a uniform hazard spectrum.



Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 12

The use of simulated ground motions should be discouraged, particularly if they are based on spectrum matching, and even more particularly if the spectrum being matched is a uniform hazard spectrum. Matching a uniform hazard spectrum is akin to subjecting the building to two (or more) simultaneous ground motions. This is not only impossible but is highly unlikely.

Whenever possible use true ground motion records. Thousands of these are available which match a variety of important conditions.

Use of Simulated Ground Motions

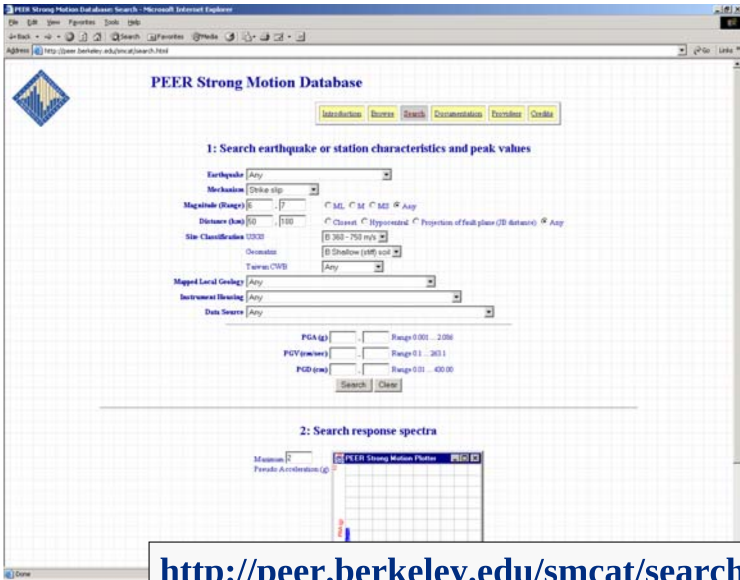
Reference:
"On the use of Design Spectrum Compatible Time Histories", by Farzad Naiem and Marshall Lew, Earthquake Spectra, Volume 11, No.1.

"Frequency domain scaled Design Spectrum Compatible Time Histories (DSCTH) are based on an erroneous understanding of the role of design spectra and can suffer from a multitude of major problems. They may represent velocities, displacements, and high energy content which are very unreliable. The authors urge extreme caution in the use of DSCTH in the design of earthquake resistant structures."

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c* - 13

This is a quote which also addresses the previous issue.

PEER Ground Motion Search Engine



http://peer.berkeley.edu/smcat/search.html

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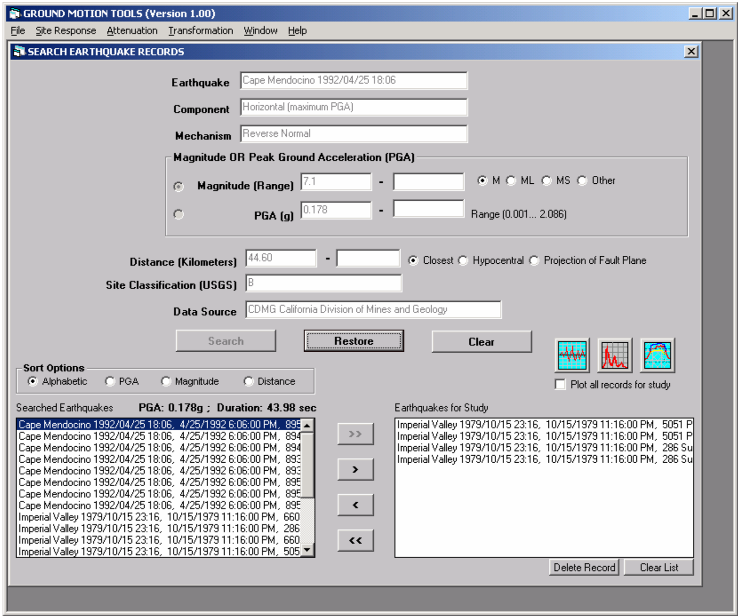
The PEER web site is one source of ground motions. A search engine is available for entering a variety of parameters. The program will provide a list of those ground motions which match the search parameters. The user may view the accelerogram or the response spectrum, and may then download the record for use in analysis.

NONLIN Ground Motion Tools (EQTOOLS)



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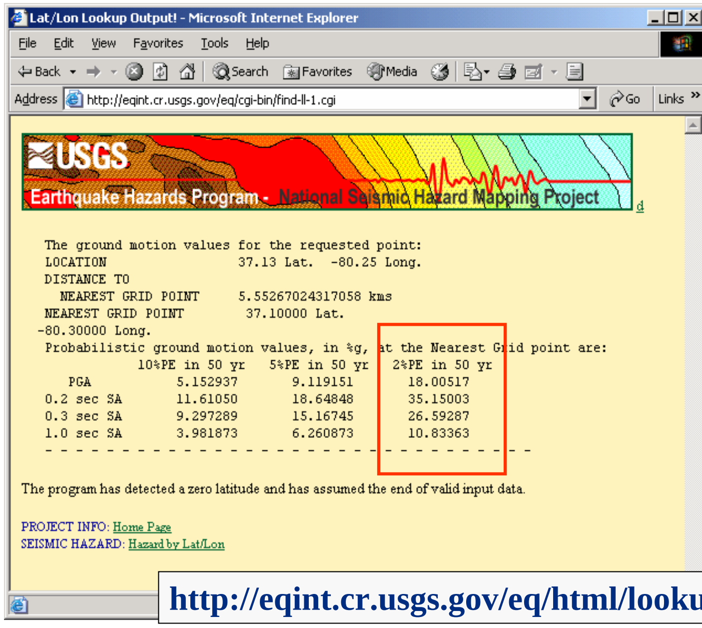
Advanced Analysis 15 – 5c - 15

A similar search engine has been developed by F. Charney and and S. Riaz at Virginia Tech. This is a stand-alone searchable data base that contains more the 2000 records, as well as a multitude of tools for evaluating the records. This program will be demonstrated during the course.

Uniform Hazard Spectrum Coordinates



http://eqint.cr.usgs.gov/eq/html/lookup.shtml

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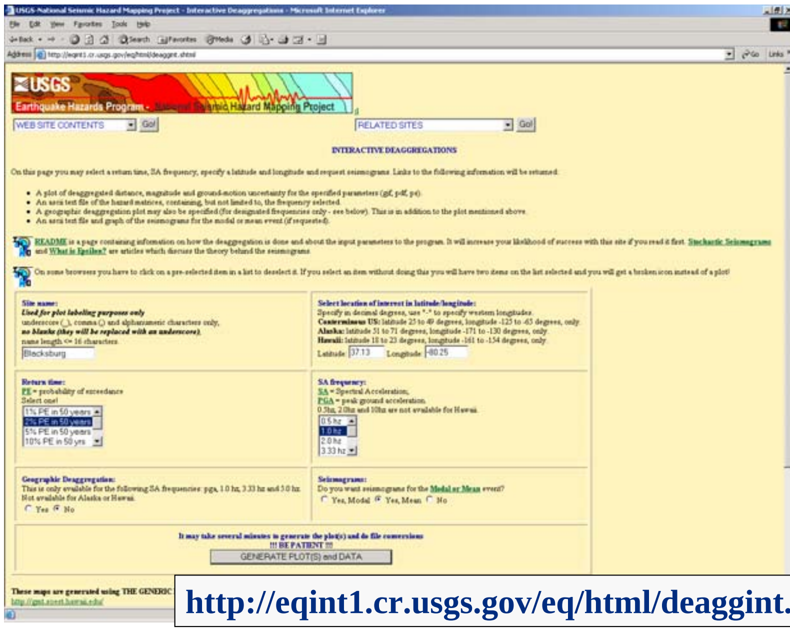
Advanced Analysis 15 – 5c - 16

Suites of artificial ground motions may be obtained from the USGS web site shown here. To obtain the records, one needs to go to the deaggregation area of the site.

Ground Motion Generator

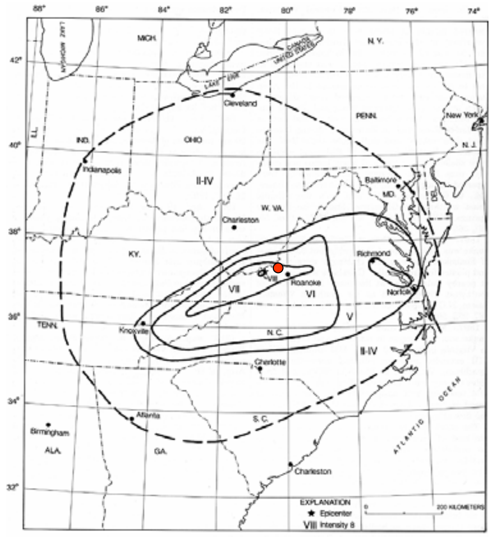


http://eqint1.cr.usgs.gov/eq/html/deaggint.shtml

FEMA   Topics in Performance-Based Earthquake Engineering   *Advanced Analysis 15 – 5c - 17*

This is the deaggregation page. We are requesting records with a 2% in 50 year probability scaled to produce pseudoaccelerations that (single point) match the 1 Hz acceleration from the 5% damped spectrum.

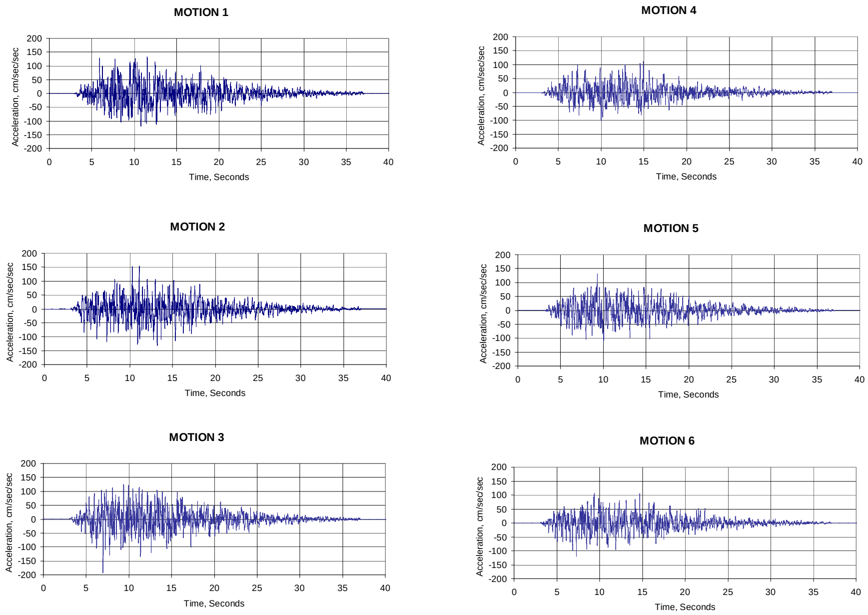
Isoseismal Map for the Giles County, Virginia, Earthquake of May 31, 1897.



Blacksburg
N 37.1
W -80.25

To illustrate, a suite of records will be obtained for Blacksburg, Virginia. Blacksburg is less than 20 KM from the epicenter of the 1897 Giles county earthquake.

Blacksburg 2%-50 Ground Motions from USGS Web Site



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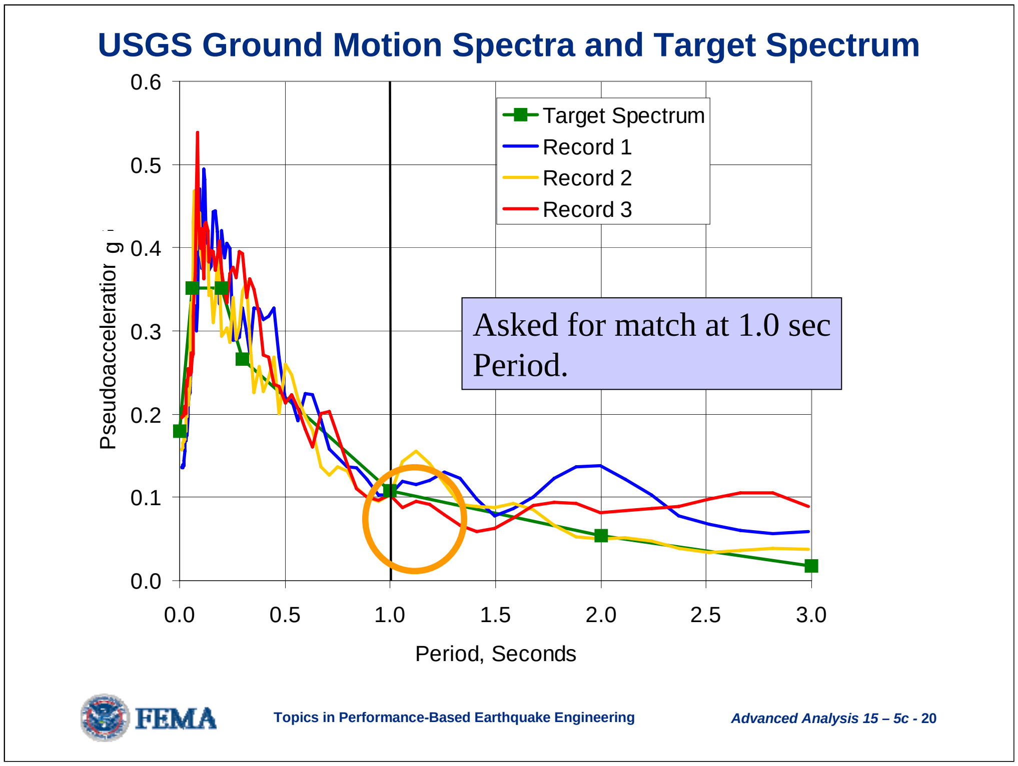
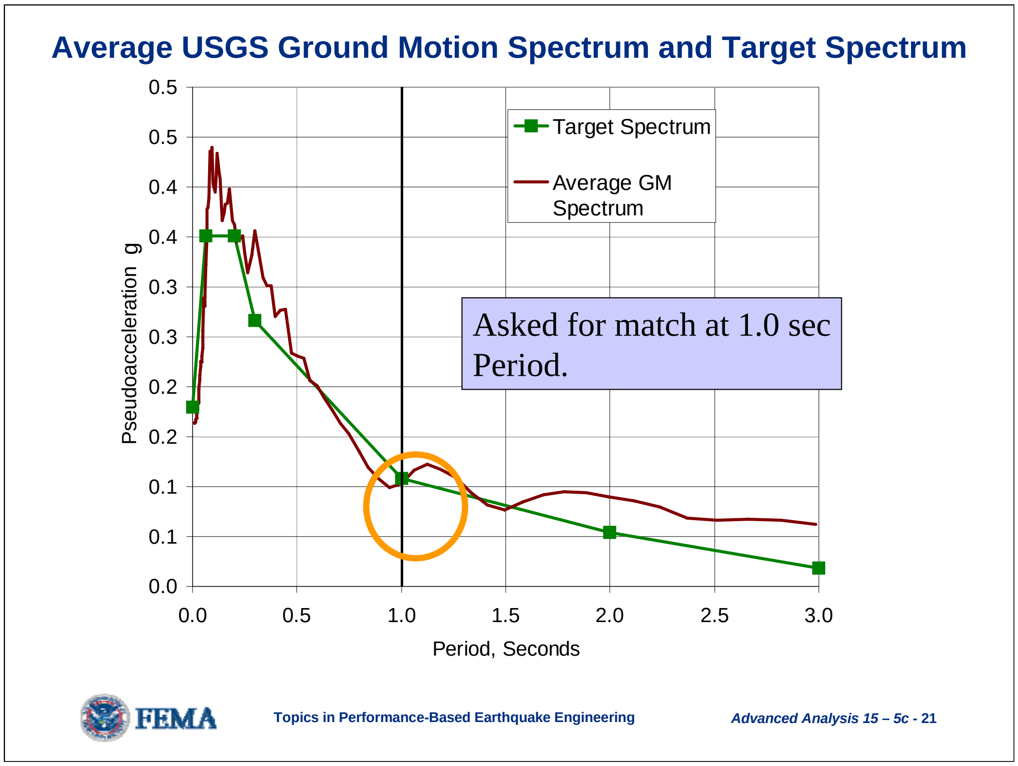
Advanced Analysis 15 – 5c - 19

These are the records obtained. Note that the peak ground accelerations are in the neighborhood of 150 cm/sec^2, or about 0.15 g.



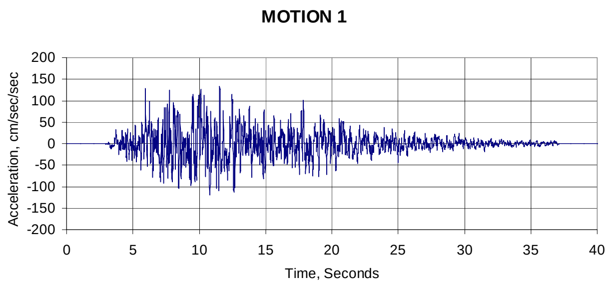
This is a plot of the 5% damped response spectra from three of the records compared to the USGS/NEHRP spectrum for site class B. As may be seen, the downloaded records have been scaled to match the USGS spectrum at the period of 1.0 seconds (1.0 Hz). Note that the peak ground accelerations (acceleration when T=0) do NOT match and that the ground motion spectra produce significantly greater accelerations at frequencies of about 0.1 to 0.2 Hz than does the USGS/NEHRP spectrum.



A better match is obtained when the average of the ground motion spectra is compared to the USGS/NEHRP spectrum.

Ground Modification Modifications

1. **Scale a given record to a higher or lower acceleration (e.g to produce a record that represents a certain hazard level)**
2. Modify a record for distance
3. Modify a record for site classification (usually from hard rock to softer soil)
4. Modify a record for fault orientation



FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 22*

There are a variety of transformations that may need to be made in a given record before it is suitable for use in response history analysis. Most of these topics are quite complex, and the modifications should be performed by an experienced seismologist. In this course basic scaling is emphasized, but the other items are addressed, specifically with respect to the new EQ-Tools program.

NEHRP Ground Motion Scaling
(2-D Analysis)

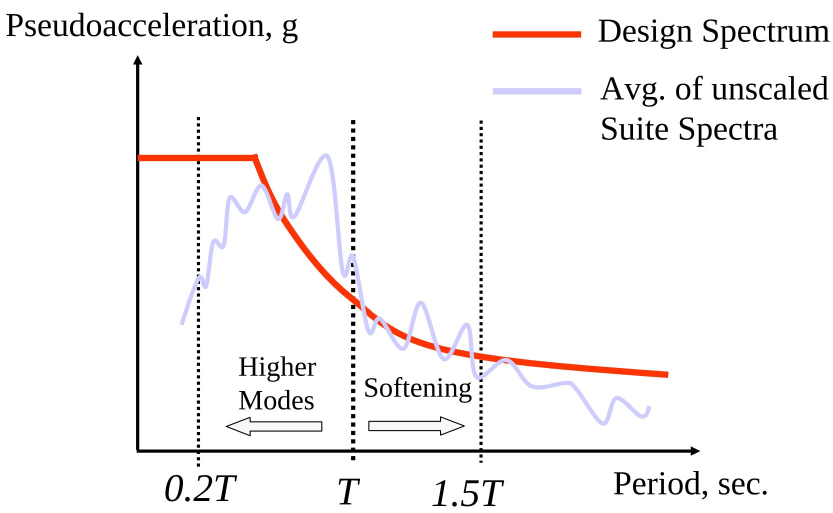
Ground motions must be scaled such that the average value of the 5% damped response spectra of the suite of motions is not less than the design response spectrum in the period range 0.2*T* to 1.5*T*, where *T* is the fundamental period of the structure.

Topics in Performance-Based Earthquake Engineering   *Advanced Analysis 15 – 5c - 23*

The 2000 and 2003 *NEHRP Recommended Provisions* (as well as ASCE7-02 and 2003 IBC) provide scaling rules that are a bit difficult to interpret, and which may produce some curious results. Different rules are provided for 2D and 3D analysis. The same rules are applied to linear and nonlinear response history analysis.

The idea behind the scaling rules is to capture the effect of period elongation associated with yielding (hence the 1.5*T* requirement), as well as to capture higher mode effects (the 0.2*T* requirement).

NEHRP Scaling for 2-D Analysis



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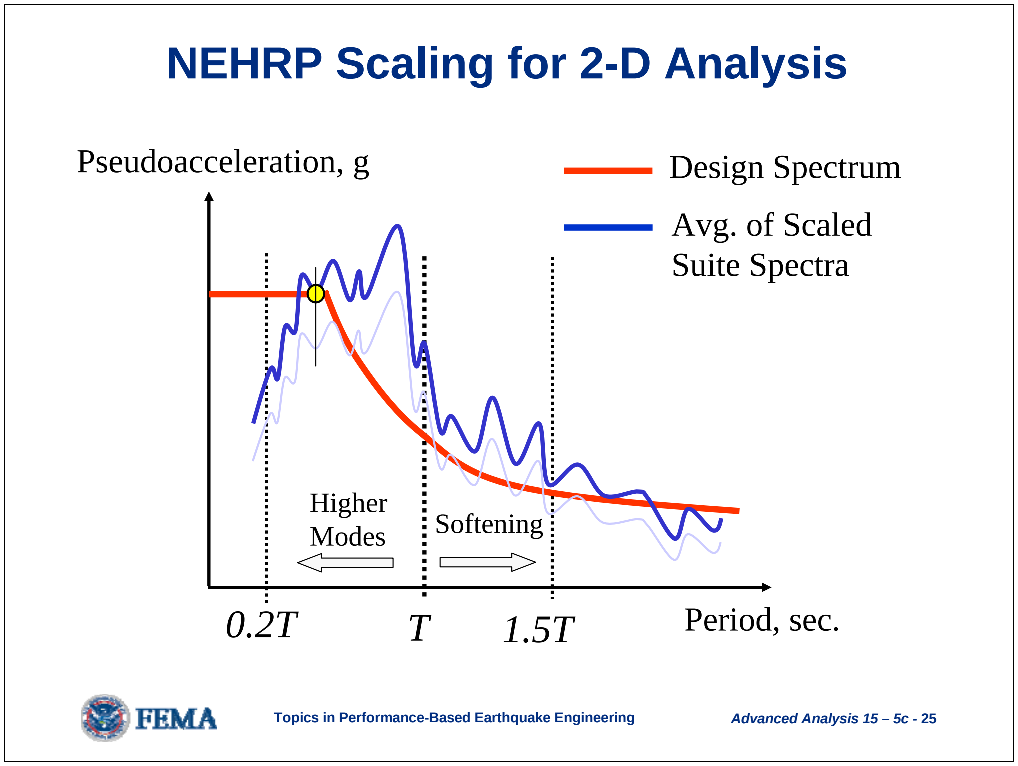
This diagram illustrates the average spectra and the code spectra before scaling. In effect, the unscaled spectra will need to be "lifted up" until all its ordinates are greater than or equal to the corresponding ordinates of the code spectra in the period range 0.2 to 1.5T.



The scaled spectra is shown in dark blue. Here, the controlling point is at a period of about 0.4T, which will definitely be a higher mode response. It seems clear that this scaling approach is extremely conservative in this case… the spectrum at the principle period of interest (T) is about 40% greater than the code spectrum at the same point.

NEHRP Ground Motion Selection and Scaling (3-D Analysis)

1. The Square Root of the Sum of the Squares of the 5% damped spectra of each motion pair (N-S and E-W components) is constructed.
2. Each pair of motions should be scaled such that the average of the SRSS spectra of all component pairs is not less than 1.3 times the the 5% damped design spectrum in the period range 0.2 to 1.5 T.

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c -* 26

Similar rules are applied for cases where a 3D analysis is used. The 1.3 factor compensates for the SRSSing of the orthogonal motion pairs.

Potential Problems with NEHRP Scaling

- A degree of freedom exists in selection of individual motion scale factors, thus different analysts may scale the same suite differently.
- The scaling approach seems overly weighted towards higher modes.
- The scaling approach seems to be excessively conservative when compared to other recommendations (e.g. Shome and Cornell)

This slide is self explanatory. The Shome and Cornell method is discussed subsequently.

How Many Records to Use?

NEHRP Recommended Provisions:

5.6.2 A suite of not less than three motions shall be used

5.6.3 If at least seven ground motions are used evaluation may be based on the average responses from the different analyses. If less than seven motions are used the evaluation must be based on the maximum value obtained from all analyses.

Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 28

The *Provisions* provide this guidance for finding the maximum response values for use in complying with performance requirements (such as inter-story drift). The use of seven or more records is recommended (by the author of this unit) because it is less conservative. However, it should be pointed out that most commercial programs provide envelope results wherein they may not provide capabilities to average across runs.

Normalization and Scaling Accelerograms For Nonlinear Analysis

Nilesh Shome and Allin Cornell
6^{th} U.S. Conference on Earthquake Engineering
Seattle, Washington, September, 1997

Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 29*

This article is an excellent reference on scaling ground motions for use in nonlinear analysis. See also the references by Cornell and Bazzurro, and Shome, Cornell, Bazzurro, and Carballo. The complete citations are given in the reference list.

Ground Motion Scaling for Nonlinear Analysis

(Shome and Cornell)

Bin:
A suite of ground motions with similar source, distance, and magnitude.

Bin Normalization:
Adjusting individual bin records to the same "intensity"

Bin Scaling:
Adjusting records from one bin (say a lower magnitude) to the intensity of the records from a different (usually higher) intensity bin.

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c* - 30

These are the principal definitions from the Shome and Cornell article, and are in pretty widespread use in the seismological community. The key differences in terms are Bin Normalization vs Bin Scaling. For example, one might have several records from similar earthquakes in the magnitude range 5 to 6. These records might be *normalized* to produce the same 1.0 second 5% damped spectral pseudoacceleration as the average of the same accelerations from the same bin. Now, assume one wants a magnitude 7 bin but has only the normalized magnitude 6 bin. These records would then be *scaled* (up) to have a magnitude consistent with a magnitude 7 earthquake.

Normalization Procedures

(Shome and Cornell)

- Normalize to PGA (NOT RECOMMENDED)
- Normalize to a Single Frequency at low damping (e.g. 2%)
- Normalize to a Single Frequency at a higher damping (e.g 5% to 20%) (RECOMMENDED)
- Normalize over a Range of Frequencies

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 31*

These are the normalization procedures recommended by Shome and Cornell. The EQ Tools program allows any of these normalization (or scaling) approaches. Note that the scaling approach adopted by the *Provisions* is not listed.

How Many Records to Use?

(Shome and Cornell)

For records normalized to first mode spectral acceleration it may typically require about **4 to 6 records** to obtain about a one sigma (plus or minus 10 to 15 percent) confidence band.

Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 32

The Shome and Cornell paper gives statistical analysis that leads to the conclusions shown here. Other normalization methods (e.g. normalization to PGA) may require twice as many records to produce results with the same level of confidence.

Can records from a low intensity bin be scaled to represent higher intensity earthquakes?

(Shome and Cornell)

When the records are scaled from one intensity level to a higher intensity there is a mild dependency of scaling on computed ductility demand. The median ductility demand may vary 10 to 20 percent for one unit change in magnitude. *The effect of scaling on nonlinear hysteretic energy demand is more significant.*

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c -* 33

Usually, one does not have any choice in this as there are very few records available from ground motions of the intensity of the MCE (Maximum Considered Earthquake). Hence, one must usually scale from a lower bin to a higher bin. Many seismologists have discouraged this in the past because of perceived differences in the frequency-amplitude characteristics of small vs large earthquakes. In the study by Shome and Cornell structures were analyzed with real magnitude M earthquakes, and the responses were compared to analyses performed with M-1 earthquakes scaled up to the M earthquakes. In the context of design, the variation in computed ductility demand is not terribly significant because a minimum amount of overstrength will reduce true ductility demand by more than 10 to 20 percent.

Recommendations (Charney):

1) Use a minimum of seven ground motions
2) If near-field effects are possible for the site a separate set of analyses should be performed using only near field motions
3) Try to use motions that are magnitude compatible with the design earthquake
4) Scale the earthquakes such that they match the target spectrum at the structure's initial (undamaged) natural frequency and at a damping of at least 5% critical.

These are the recommendations of the principal author of this topic (Finley Charney). Note that Charney is *not* a seismologist.

Ground Modification Modifications

1. **Scale a given record to a higher or lower acceleration (e.g to produce a record that represents a certain hazard level)**
2. Modify a record for distance (SRL Attenuation Issue)
3. Modify a record for site classification, usually from hard rock to softer soil. (WAVES by Hart and Wilson)
4. Modify a record for fault orientation (Somerville, et al)

See Also: *Ground Motion Evaluation Procedures for Performance Based Design*, by J.P. Stewart, et al, PEER Report 2001/09

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c* - 35

Aside from normalization/scaling, there are other factors that must be considered in obtaining ground motions for nonlinear analysis. While most of these are beyond the scope of this Topic, it is noted that the EQTools provides utilities for each of the three additional items listed here.

The modifications of record for distance may be based on a variety of attenuation relationships that are built into the program. The attenuation relationships built into the program have been described in some detail in the Seismological Research Letters, Volume 68, No. 1, January/February 1997.

The modification for site effects is based on the Waves Program developed by Hart and Wilson (Simplified Earthquake Analysis of Buildings Including Site Effect, Report UCB/SEMM 89/23, UC Berkeley.)

Modification for fault orientation is a simple transformation that corrects for the observed differences between strike-normal and strike-parallel components of near-fault ground motions at periods longer than about 0.5 seconds. See, for example, Somerville, et al, 1997.

Damage Prediction

Performance based design requires a quantification of the damage that might be incurred in a structure.

The “damage index” must be calibrated such that it may predict and quantify damage at all performance levels.

While inter-story drift and inelastic component deformation may be useful measures of damage, a key characteristic of response is missing… the effect of the duration of ground motion on damage.

A number of different damage measures have been proposed which are dependent on duration.

FEMA Topics in Performance-Based Earthquake Engineering Advanced Analysis 15 – 5c - 36

In performance based engineering, the goal is to limit damage (maintain functionality) at the prescribed hazard levels. A large number of damage measures have been proposed by various researchers. These range from simple inter-story drift to complex cumulative energy based indices. However, any true damage measure must be duration dependent. Simple drift as a damage measure is not duration dependent, and hence, is not appropriate. Interestingly, most of the code-based damage measures are based on simple drift.

Damage Prediction

Park and Ang (1985)

$$DI_{PA} = \frac{u_{max}}{u_{cap}} + \lambda \frac{E_H}{u_{cap} F_y}$$

u_{max} = maximum attained deformation

u_{cap} = monotonic deformation capacity

E_H = hysteretic energy dissipated

F_y = monotonic yield strength

λ = calibration factor

See Reference List for Additional Info on Damage Measures

This is one of the earliest duration-dependent damage measures or damage Indices. The first term in the sum is effectively a simple ductility demand ratio. The second term is a measure of the cumulative energy dissipated compared to the energy dissipated in 1/4 of a full reversed yield excursion. The term λ is a calibration factor.

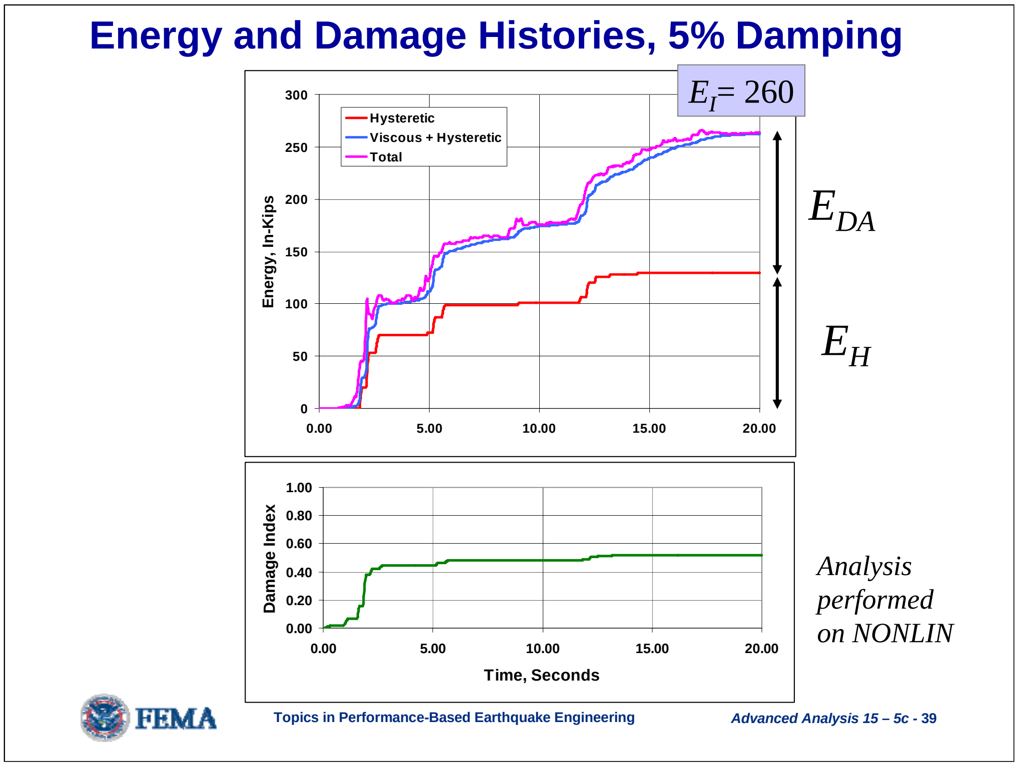
λ is calibrated such that a DI_{PA} of 1.0 is indicative of insipient collapse. Indices less than about 0.2 would indicate little or no damage.

As stated herein, a performance criterion might be based on the maximum damage index attained in any component. This approach may put undue influence on a single component. To remove this potential problem several weighted average approaches have been forwarded. See for example Mehanny and Deirlein (2000).

Energy Balance

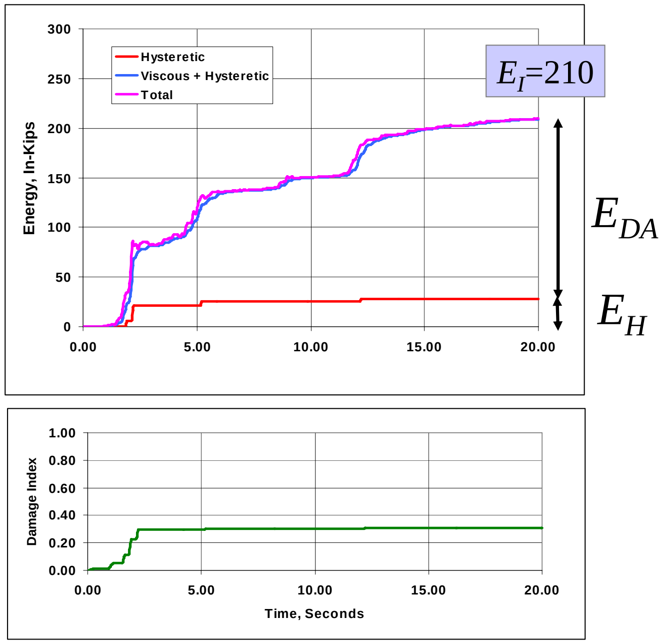
$$E_I = E_S + E_K + (\underbrace{E_{DI}}_{\text{Inherent Damping}} + \underbrace{E_{DA}}_{\text{Added Damping}}) + \overbrace{E_H}^{\text{Hysteretic Energy}}$$

This slide simply illustrates the various sources of energy absorption/dissipation in a structure. The more hysteretic energy moved into added damping, the lower the damage. This is a tremendous selling point for added damping systems. This concept is illustrated on the following two slides.



This slide represents the response of a SDOF system responding to the 1940 Imperial Valley Earthquake. The maximum energy input is 260 in-kips. This is shared almost equally by damping and hysteretic effects. The maximum DI is about 0.5, which would be indicative of significant damage. Note that the hysteretic energy remains constant at T=15 seconds, hence, damage is not accumulated beyond that point.

Energy and Damage Histories, 20% Damping



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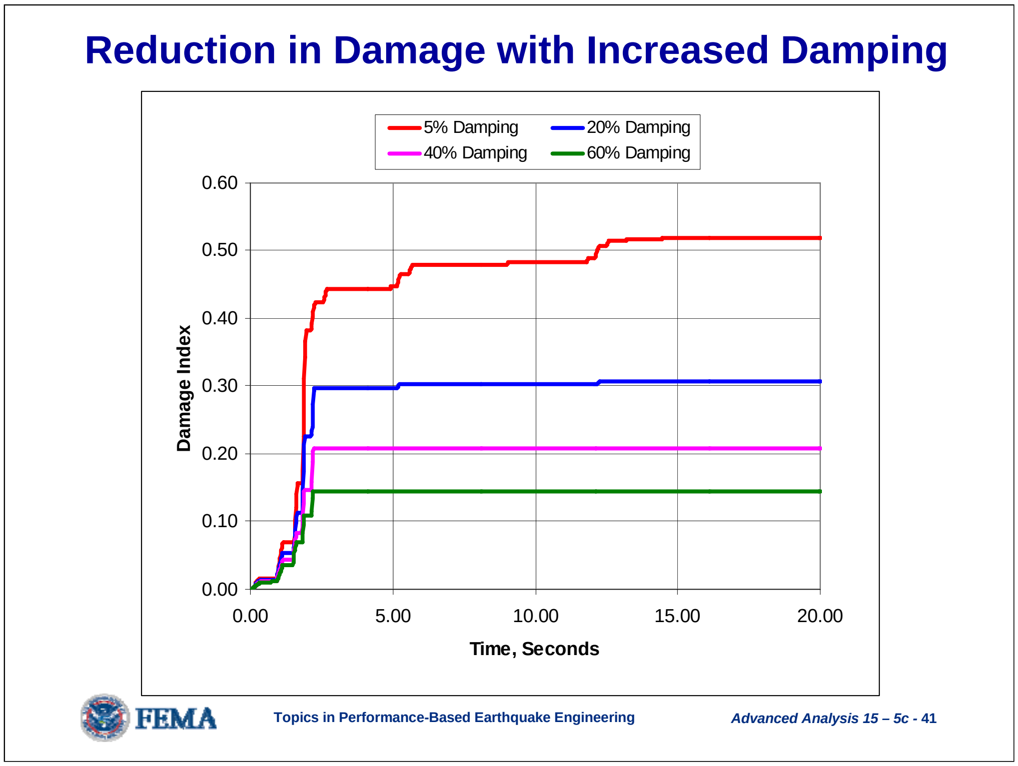
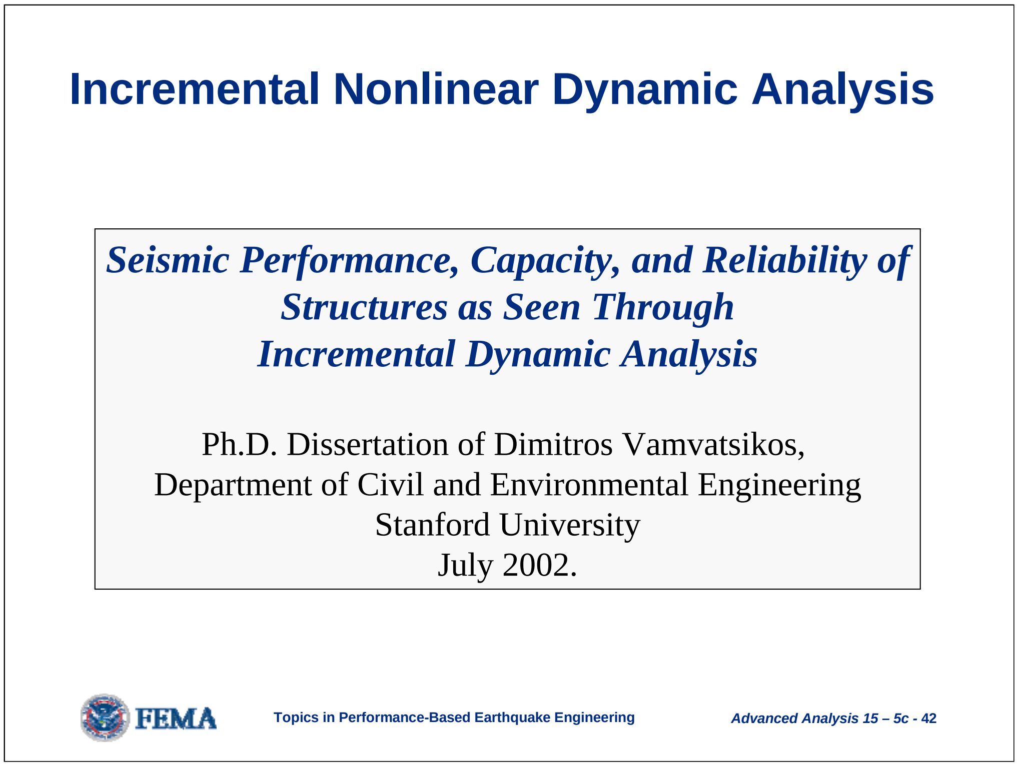
Now damping has been increased to 20% critical by adding a viscous fluid device. No additional stiffness has been provided. The total energy input demand has reduced to 210 in-k. More significantly, the hysteretic energy demand has reduced significantly, leading to a reduced DI of about 0.3. The structure would still be damaged at this point, but much less than for a DI of 0.5. Note also that the increase in damage effectively terminates at about 12 seconds into the response.

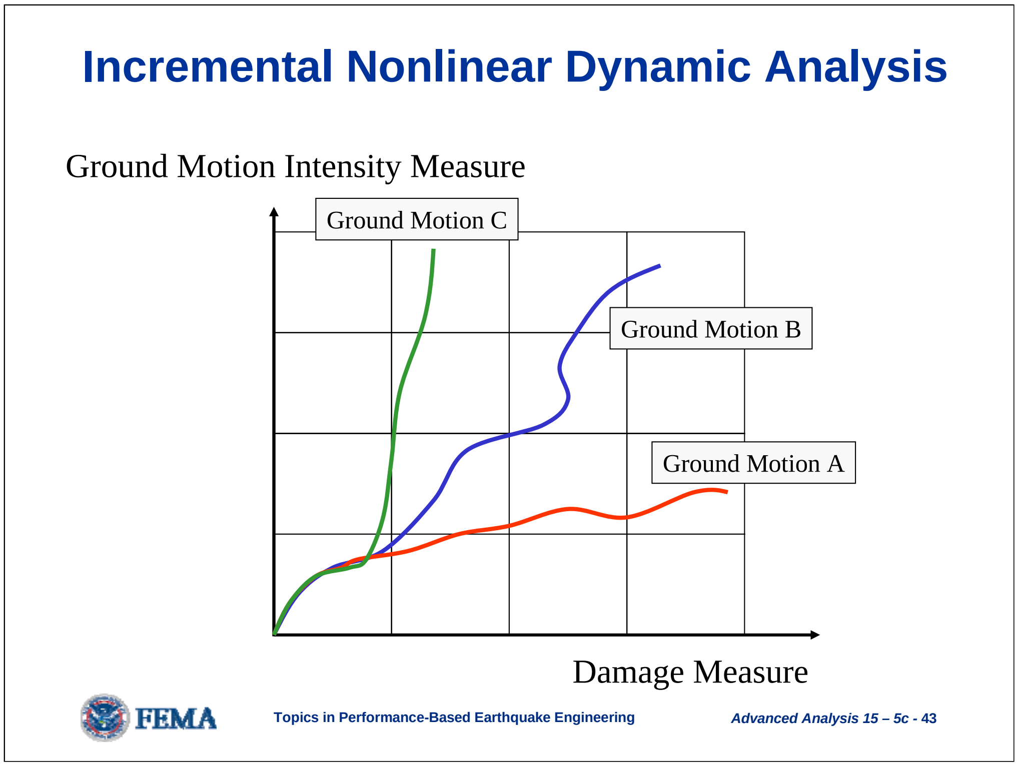


This slide show the DI histories that result from a variety of damping values. Increasing damping to 60 percent critical (probably not physically possible) has reduced the maximum DI from 0.52 to about 0.14. The lower number is indicative of a nearly elastic response.



This is the principal reference for the slides on incremental nonlinear dynamic analysis.

The next several slides cover the subject of Incremental Dynamic Analysis. This is a relatively new approach that provides very valuable information on the response of structures to ground motions, and particularly, provides information on the sensitivity of response (some Damage Measure) to variations in ground motion, ground motion intensity, or structural characteristics (e.g. stiffness, strength, hysteretic behavior, damping…)

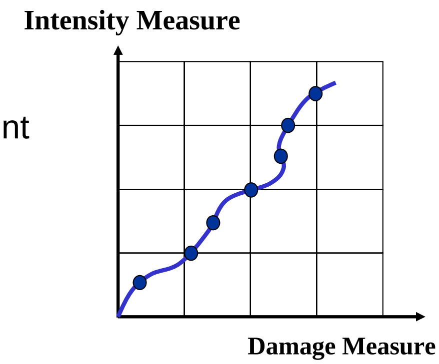


This slide shows three hypothetical IDA curves representing the response of a system to thee different (but equivalently scaled) ground motions. Differences in the three curves are characteristic of IDA analysis, as will be discussed later.

Incremental Nonlinear Dynamic Analysis

An *IDA study* is produced by subjecting a single structure to a series of time history analyses, where each subsequent analysis uses a higher ground motion intensity.

An *IDA Curve* is a plot of a damage measure (DM) versus the ground motion intensity (IM) at which it occurred.



FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 44*

These are the basic definitions for an IDA analysis.

The IDA curve is developed by subjecting a structure to a single ground motion, but reanalyzing for that ground motion at increasing intensities. For example, if the 5% damped MCE spectral pseudoacceleration at the structure's first mode period is 0.3 g, response history analyses might be performed for the structure subjected to the earthquake at 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, 0.35, 0.40, 0.45, 0.50, 0.55, and 0.60 g. These different ground motion intensities are called the Intensity Measure.

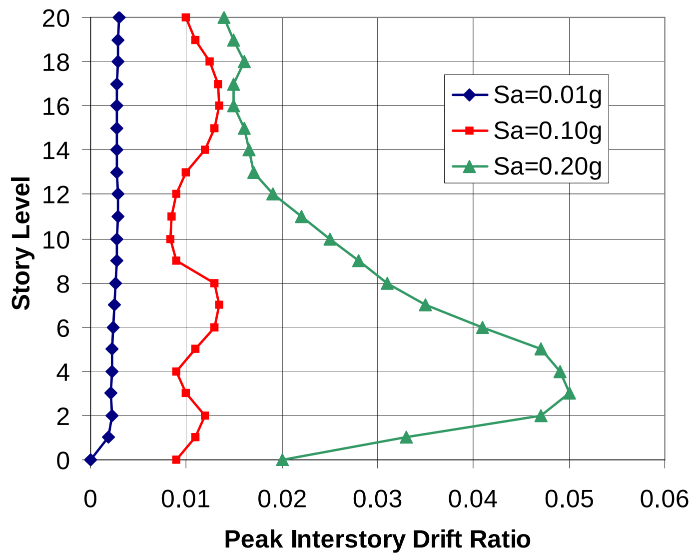
For each intensity measure, the peak Damage Measure would be recorded. The IDA curve is a plot of the intensity versus damage measure.

Damage measures can be any computed quantity, such as maximum inter-story drift, maximum plastic hinge rotation, maximum Park and Ang Damage Index, maximum Base Shear, and so on.

Note that it may take 100 or more response history analyses to produce an IDA curve of sufficient resolution. Given that IDA curves may need to be produced for an entire suite of ground motions, it is easy to see that the procedure is very computationally expensive.

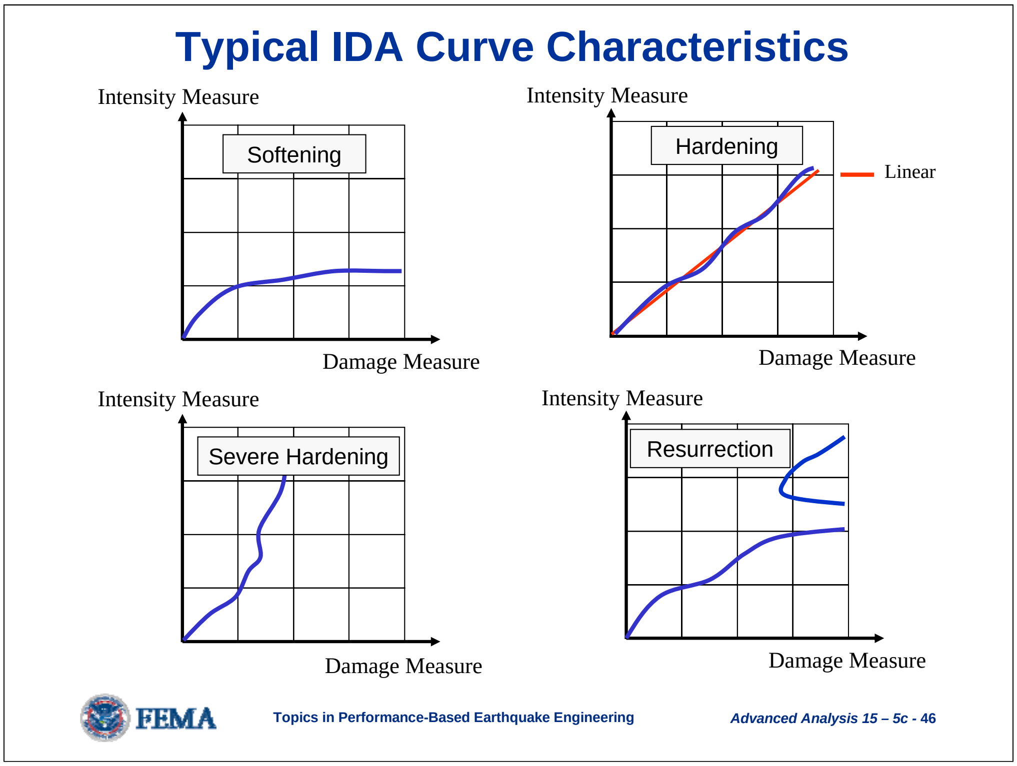
IDA Results for a Particular Ground Motion

(after Vamvatsikos and Cornell)



FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 45*

Here, IDA results are presented in a different form. The Damage Measure is interstory drift, and these measures are plotted versus the story level at which the drift was obtained. The different curves represent different Intensity Measures. It is easy to see from this plot that the increment in ground motion intensity from 0.1 to 0.2 g had a profound effect on the drifts of the lower stories.



IDA produces an interesting variety of behaviors in structures. The four main observable behaviors are shown here.

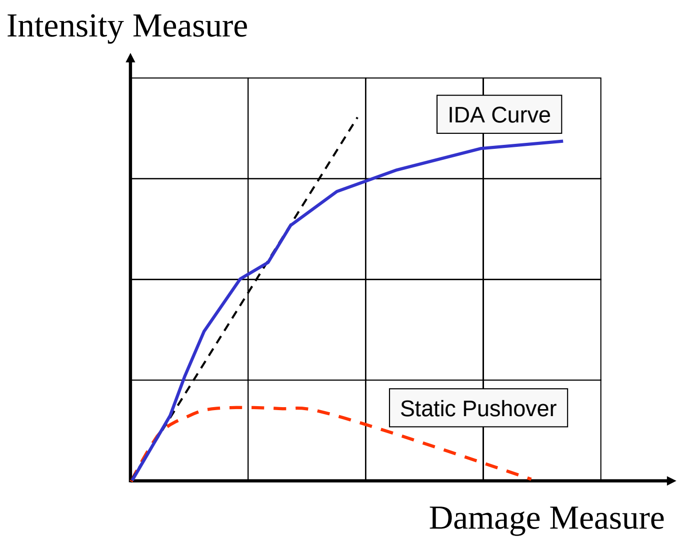
A softening system is one that has greatly increasing displacements with slight increases in ground motion intensity. This is typically due to dynamic instability.

A hardening system has an essentially linear IDA curve. (Note that this structure yields under the higher intensity ground motions. If the structure remained linear the IDS curve would be a straight line as indicated by the red line, which it is NOT doing here). A structure with a hardening IDA appears to be following the “equal displacement” rule, that is the elastic and inelastic displacements are essentially the same for a yielding structure.

A severe hardening system is one in which the displacements grow very slowly, cease to grow, or even *decrease* with increasing ground motion intensity.

Resurrection is the remarkable phenomenon that a structure which collapses under one intensity of ground motion actually survives a greater level of ground motion. This behavior is due to phasing of yielding in the structure and acceleration/velocity/displacement pulses in the ground motion.

Typical IDA Curve Characteristics



FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 47*

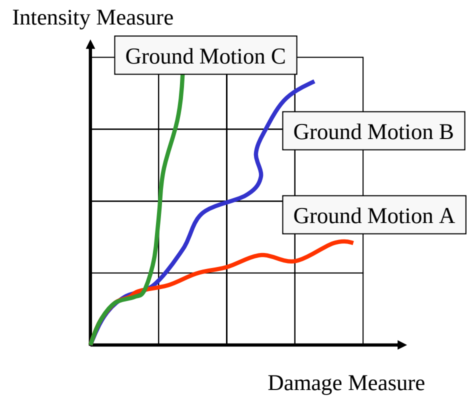
The IDA curve and the Static Pushover curve are typically dissimilar as shown here. However, techniques (including a computer program called SPO2IDA) have been developed to create simplified (and quite accurate) IDA curves from basic pushover shapes. See Chapter 4 of the dissertation of Vamvatsikos for details.

Incremental Nonlinear Dynamic Analysis

(using Multiple Ground Motions)

Usually, a study compares the response of the structure to a suite of ground motions.

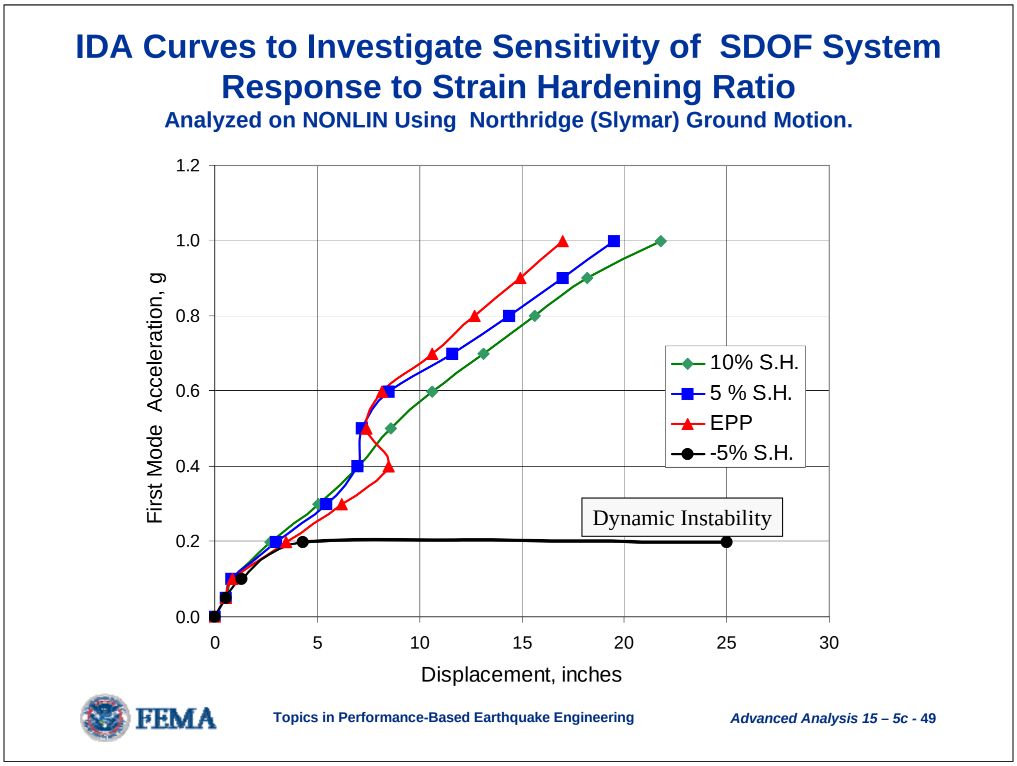
An IDA study may also be used to assess the effect of a design change (or uncertainty) on the response of a structure to a particular ground motion.



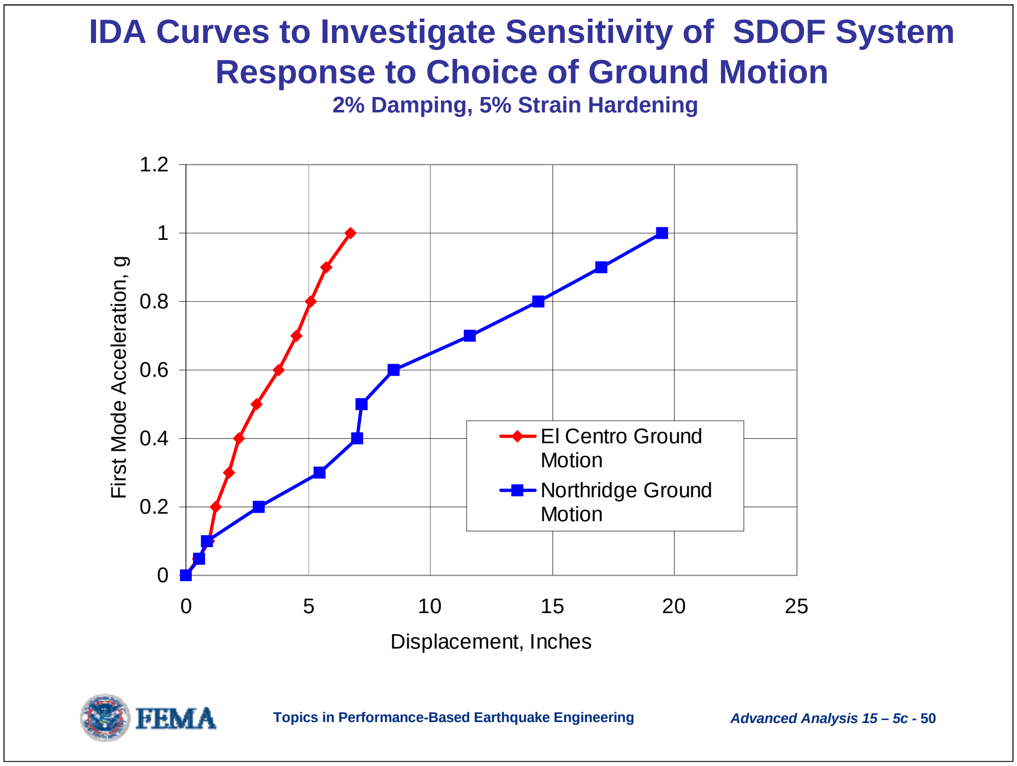
Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 48

IDA curve for the same structure subjected to similar (scaled) ground motions can be quite different. Also, IDA curves for variations of the same structure to a single ground motion (of varying intensity) can be quite different.

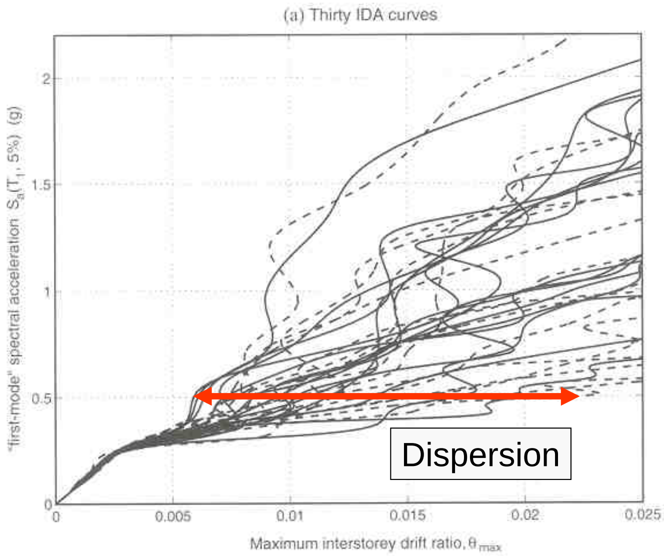


In this example a single structure has been subjected to a single ground motion. The structural parameters have been changed to reflect different strain hardening ratios. For the system with a negative ratio, dynamic instability ensues at a ground motion intensity of about 9.20. Note how the EPP response started to deviate from the "elastic" response (moving to the right) but then wove to the left, eventually developing lower displacement than the systems with stiffer strain hardening.



Here, the same system was subjected to two different (but equivalently scaled) ground motions. The response to El Centro is strongly hardening, and the response to Northridge is effectively linear, with a little weaving.

A Family of IDA Curves of the Same Building Subjected to Thirty Earthquakes



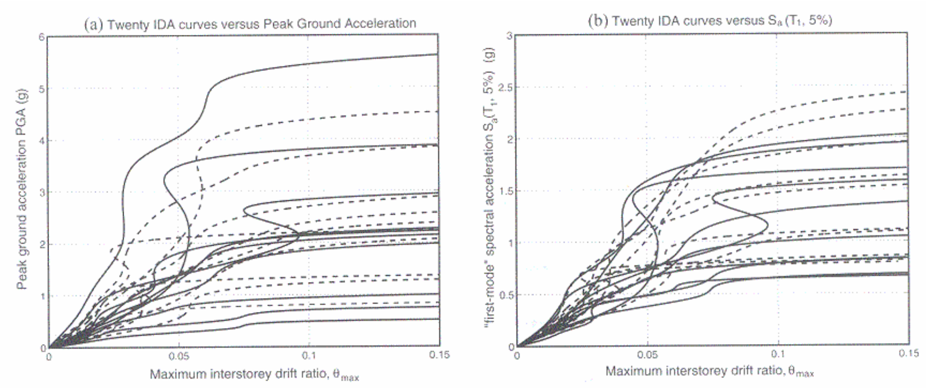
Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 51

This plot, from Cornell and Vamvatisikos, shows IDA curves for a single structure subjected to 30 different ground motions. All of the systems appear to have the same general behavior up to an intensity of about 0.25g, after which all bets are off. At intensities of 0.5 g, the variations in response are huge. The variation in response at a given intensity is referred to as dispersion. The huge dispersion here is characteristic of IDA analysis. If the actual ground motion intensity was 0.5g, how would one determine if the design is appropriate?

IDA Curves of the Same Building Subjected to Suite of Earthquakes

NORMALIZED to PGA NORMALIZED to SA



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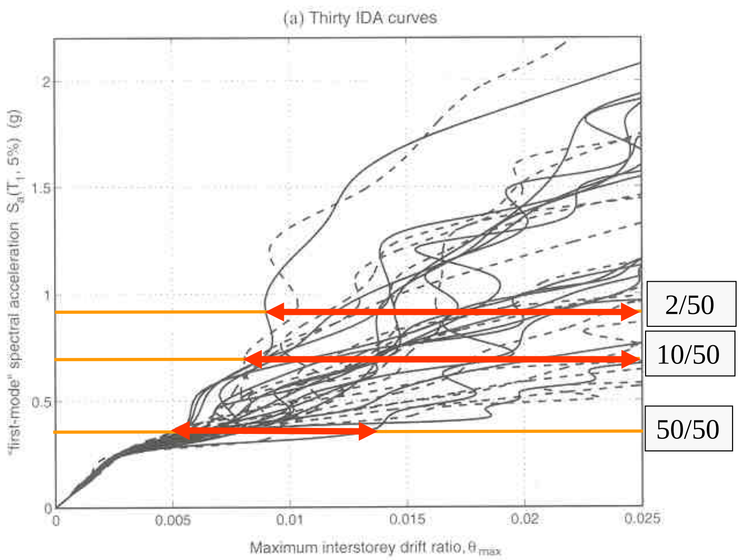
Here, sets of IDA were performed using two different scaling techniques. On the left, scaling was normalized to PGA. On the right, scaling was based on 5% damped spectral pseudoacceleration at the structure's first mode natural frequency. The variation in dispersion is quite remarkable, with far less dispersion being evident on the right. Other items, such as damage measure, have an effect on dispersion. It would seem that limiting dispersion would be desirable.

A Family of IDA Curves of the Same Building Subjected to Thirty Earthquakes



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Here, the IDAs based on spectrum scaling at the first mode frequency are shown with different levels of probability of the ground motions labeled. For the 50 percent probability in 50 year ground motion (100 MRI) there is relatively little dispersion. For the 10 and 2 percent in 50 year probabilities (500 and 2500 mean return interval), dispersion is increased, and is effectively infinite if dynamic instability ensued. Again, how is one to make design decisions given the apparent dispersion. Does this nullify the use of response history analysis as a viable design tool?

Incremental Nonlinear Dynamic Analysis

- Use of IDA shows the **EXTREME** sensitivity of damage to ground motion intensity, as well as the **EXTREME** sensitivity of damage to the chosen ground motion.
- Dispersion in multiple ground motion IDA may be reduced by scaling each base ground motion to a target spectral intensity computed at the structure's fundamental frequency of vibration.
- Even with such scaling, it is clear that PBE assessments based on response history analysis is problematic if carried out in a purely deterministic framework. Probabilistic methods must be employed to adequately handle the randomness of the input and the apparent "chaos" in the results.

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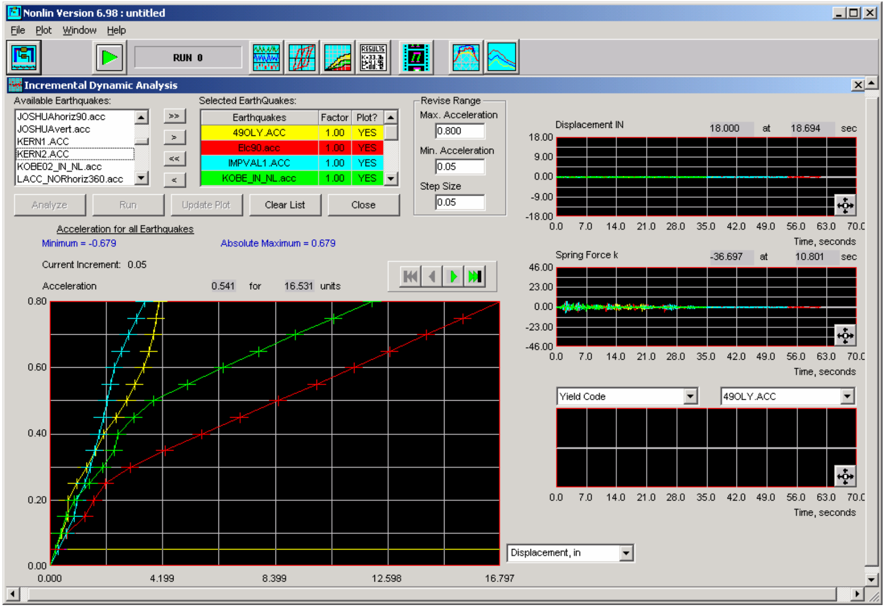
Self explanatory summary.

NONLIN Version 7 IDA Tool



Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 55

It is noted here that IDA has been added to the NONLIN program in Version 7. This would be a good point to demonstrate the use of this tool.

Probabilistic Approaches to Performance-Based Engineering

The Most Daunting Task: Identifying and Quantifying Uncertainties

Demand Side (Ground Motion)

1) Magnitude
2) Source Mechanism
3) Wave Propagation Direction
4) Attenuation
5) Site Amplification
6) Frequency Content
7) Duration
8) Sequence (foreshocks, aftershocks)

…

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c* - 56

One of the main advantages of the probabilistic approach is that is has the capability to directly account for a wide range of uncertainties inherent in earthquake engineering. A list of some of the types of uncertainties is listed here. Note that these uncertainties are on the demand (load) side of the equation and have nothing to do with actual response prediction.

Probabilistic Approaches
The Most Daunting Task:
Identifying and Quantifying Uncertainties

Capacity Side (Soil/Foundation/Structure Behavior)

1) Strength
2) Stiffness
3) Inherent Damping
4) Hysteretic Behavior
5) Gravity Load
6) Built-in Imperfections

…

Analysis Uncertainties

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 57*

This is a list of uncertainties on the capacity side… those having to do with predicting the response of the system. Another set of uncertainties is related to the analysis side.

PEER's Probabilistic Framing Equation

$$\lambda(DV) = \iint G(DV|DM)\ |dG(DV|IM)|\ |d\lambda(IM)|$$

$\lambda(DV)$ Likelihood of exceeding a certain limit state

IM Intensity Measure
DM Damage Measure
DV Decision Variable

PEER (The Pacific Earthquake Engineering Research Center) is working to develop a probabilistic framework for performance based earthquake engineering. Generally, we are trying to calculate the likelihood (probability) of exceeding some damage state (say interstory drift) given a variety of parameters and associated uncertainties.

Probabilistic Approaches: FEMA 350

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx$$

$P(D > PL)$	Probability of damage exceeding a performance level in a period of *t* years
$P_{D>PL}(x)$	Probability of damage exceeding a performance level given that the ground motion intensity is level *x*, as a function of *x*.
$h(x)dx$	Probability of experiencing a ground motion intensity of level (x) to ($x+dx$) in a period of *t* years

Topics in Performance-Based Earthquake Engineering   *Advanced Analysis 15 – 5c* - 59

FEMA 350 has developed a probabilistic approach for the design of steel frame structures. Its “equation” appears somewhat simpler (and is simpler to understand) than the PEER equation.

Probabilistic Approaches: FEMA 350

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx$$

Simplified Method

Detailed Method

Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 60

A simplified methodology has been provided in FEMA 350 to perform the probabilistic analysis. This is briefly described next.

Probabilistic Approaches: FEMA 350

$$\lambda = \frac{\gamma \, \gamma_a \, D}{\phi C}$$

λ	Capacity to Demand Ratio
γ	Demand Variability Factor
γ_a	Analysis Uncertainty Factor
C	Tabulated Capacity for the Component
ϕ	Capacity Resistance Factor
D	Calculated Demand for the Component
β_{UT}	Total Coefficient of Variation

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c* - 61

In the FEMA 350 approach, the engineer establishes a “confidence” that a certain level of damage will not be exceeded for the design ground motion. Under this confidence level the computed capacity to demand ratio must be less that or equal to the limiting value λ.

Probabilistic Approaches: FEMA 350

Table 4-7
Recommended Minimum Confidence Levels

Behavior	Performance Level	
	Immediate Occupancy	Collapse Prevention
Global Interstory Drift	50%	90%
Local Interstory Drift	50%	50%
Column Compression	50%	90%
Splice Tension	50%	50%

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 62*

The minimum confidence levels are based on the damage measure and on the limit state. For example, for Collapse Prevention, one must be 90% confident that the global interstory drift damage measure will not be exceeded.

Probabilistic Approaches: FEMA 350

Table 4-8
Interstory Drift Angle Analysis Uncertainty Factor γ_a

	Analysis Procedure	LSP		LDP		NSP		NDP	
	System Characteristic	I.O	C.P.	I.O	C.P.	I.O	C.P.	I.O	C.P.
Special	Low Rise (<4 stories)	0.94	0.70	1.03	0.83	1.13	0.89	1.02	1.03
Special	Mid Rise (4-12 stories)	1.15	0.97	1.14	1.25	1.45	0.99	1.02	1.06
Special	High Rise (> 12 stories)	1.12	1.21	1.21	1.14	1.36	0.95	1.04	1.10
Ordinary	Low Rise (<4 stories)	0.79	0.98	1.04	1.32	0.95	1.31	1.02	1.03
Ordinary	Mid Rise (4-12 stories)	0.85	1.14	1.10	1.53	1.11	1.42	1.02	1.06
Ordinary	High Rise (> 12 stories)	0.80	0.85	1.39	1.38	1.36	1.53	1.04	1.10

FEMA Topics in Performance-Based Earthquake Engineering Advanced Analysis 15 – 5c - 63

The interstory drift angle uncertainty factor is a function of the method of analysis, governing limit state, the type of system, and the number of stories. Note that it is difficult to discern a trend in the tabulated values.

Probabilistic Approaches: FEMA 350

Table 4-9
Interstory Drift Angle Demand Variability Factor γ

	Building Height	γ I.O.	γ C.P.
Special	Low Rise (< 4 stories)	1.5	1.3
Special	Mid Rise (4-12 stories)	1.4	1.2
Special	High rise (>12 stories)	1.4	1.5
Ordinary	Low Rise (< 4 stories)	1.4	1.4
Ordinary	Mid Rise (4-12 stories)	1.3	1.5
Ordinary	High rise (>12 stories)	1.6	1.8

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Here is the drift angle demand variability factor. Again, it is difficult to discern a trend.

Probabilistic Approaches: FEMA 350

Table 4-10
Global Interstory Drift Angle Capacity Factors (C) and Resistance Factors (ϕ)

	Building Height	I.O.		C.P.	
		C	ϕ	C	ϕ
Special	Low Rise (<4 stories)	0.02	1.00	0.10	0.90
Special	Mid Rise (4-12 stories)	0.02	1.00	0.10	0.85
Special	High Rise (> 12 stories)	0.02	1.00	0.09	0.75
Ordinary	Low Rise (<4 stories)	0.01	1.00	0.10	0.85
Ordinary	Mid Rise (4-12 stories)	0.01	0.90	0.08	0.70
Ordinary	High Rise (> 12 stories)	0.01	0.85	0.06	0.60

FEMA

Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 65

This is the global inter-story drift angle capacity and resistance factor table. Table 4-12 (not shown here) gives similar values for local connection response.

Probabilistic Approaches: FEMA 350

Table 4-11
Uncertainty Coefficient β_{UT} for Global Interstory Drift Evaluation

	Building Height	Perf. Level	
		I.O.	C.P.
Special	Low Rise (< 4 stories)	0.20	0.30
Special	Mid Rise (4-12 stories)	0.20	0.40
Special	High rise (>12 stories)	0.20	0.50
Ordinary	Low Rise (< 4 stories)	0.20	0.35
Ordinary	Mid Rise (4-12 stories)	0.20	0.45
Ordinary	High rise (>12 stories)	0.20	0.55

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This is the uncertainty coefficient for global interstory drift evaluation.

Probabilistic Approaches: FEMA 350

Table 4-6
Confidence Levels for Various Values of λ and β_{UT}

Confidence Level	10	20	30	40	50	60	70	80	90	95	99
λ for $\beta_{UT} = 0.2$	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
λ for $\beta_{UT} = 0.3$	1.68	1.48	1.34	1.24	1.14	1.06	0.98	0.89	0.78	0.70	0.57
λ for $\beta_{UT} = 0.4$	2.12	1.79	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.51
λ for $\beta_{UT} = 0.5$	2.76	2.23	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
λ for $\beta_{UT} = 0.6$	3.70	2.86	2.36	1.99	1.72	1.48	1.25	1.03	0.80	0.64	0.43

FEMA Topics in Performance-Based Earthquake Engineering Advanced Analysis 15 – 5c - 67

If one has a target confidence level of 90% and an Uncertainty coefficient of 0.4, the limiting capacity to demand ratio is 0.76. Using this value, one can go on to determine the limiting demand as shown in the next slide.

One could alternatively compute the capacity to demand ratio from an analysis, and then work out the corresponding confidence. If the confidence was too low a revised design would be required.

Probabilistic Approaches: FEMA 350

Example Calculations for 4-12 Story Frame (DL is "Allowable" Interstory Drift Limit)

Type	PERF	Analysis	Confidence	γ	γ_a	ϕ	C	β_{UT}	λ	DL
SPECIAL	IO	NSP	50%	1.4	1.45	1	0.02	0.2	1.06	0.0104
SPECIAL	IO	NDP	50%	1.4	1.02	1	0.02	0.2	1.06	0.0148
SPECIAL	CP	NSP	90%	1.2	0.99	0.85	0.1	0.4	0.76	0.0544
SPECIAL	CP	NDP	90%	1.2	1.06	0.85	0.1	0.4	0.76	0.0508
ORDINARY	IO	NSP	50%	1.3	1.11	0.9	0.01	0.2	1.06	0.0066
ORDINARY	IO	NDP	90%	1.3	1.02	0.9	0.01	0.2	1.06	0.0072
ORDINARY	CP	NSP	50%	1.5	1.42	0.7	0.08	0.45	0.765	0.0201
ORDINARY	CP	NDP	90%	1.5	1.06	0.7	0.08	0.45	0.765	0.0269

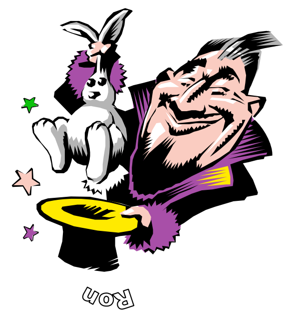
FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 68*

For example for a 4-12 story Special Moment Frame analyzed using the nonlinear dynamic procedure, and with a collapse prevention confidence requirement of 90 percent, the computed interstory drift should be limited to 0.0508 (a startlingly large number). Note that far less drift is allowed for an Ordinary Moment Frame of the same height with the same limit state and confidence requirement.

Problem with FEMA 350 Approach?

Even though the method provides the owner a "Level of Confidence" that a certain performance criteria will be met, the engineer is likely to be bewildered by the arrays of coefficients. Hence, it is difficult for the engineer to obtain a feel for the validity of the results.

Given this, how confident is the engineer with the value of confidence provided?

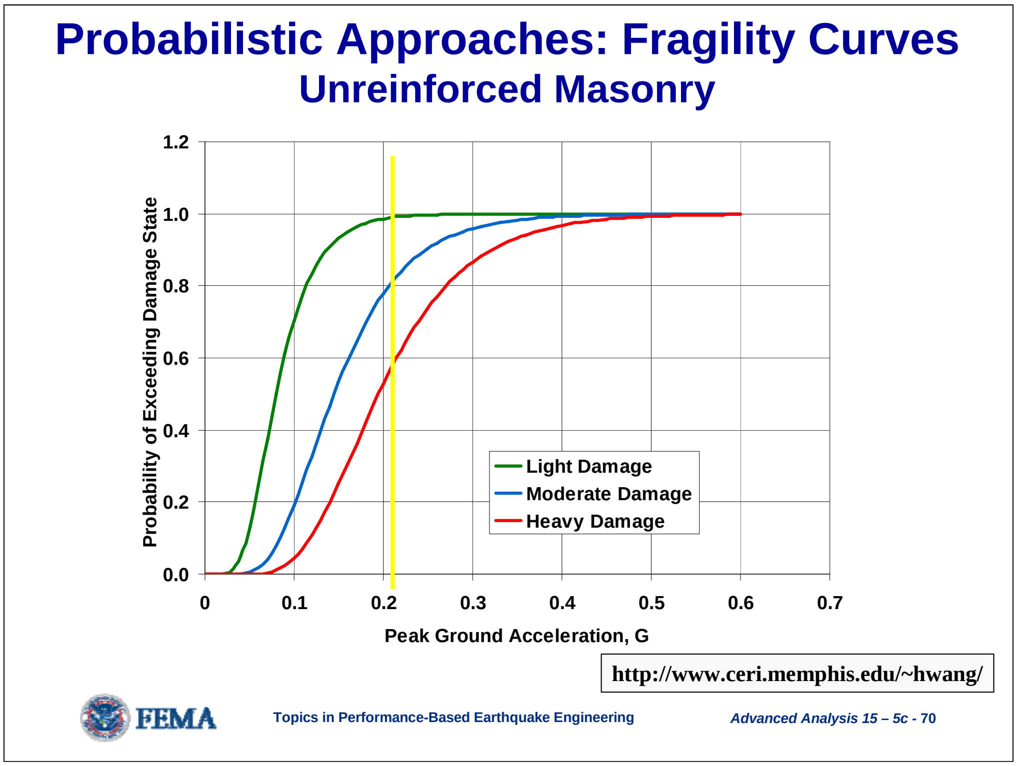


FEMA

Topics in Performance-Based Earthquake Engineering

Advanced Analysis 15 – 5c - 69

The author of this topic (Charney) feels that the FEMA 350 method is a good start, but that more needs to be done to make the procedure more understandable to the engineer.

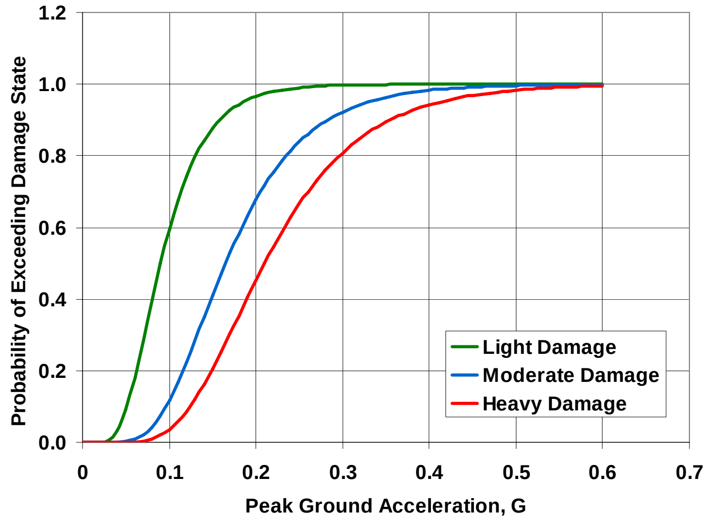


Probabilistic approaches may also be used to establish a set of “Fragility Curves” for a given structural system. Here, for example, are fragility curves for unreinforced masonry structures used as fire houses. The larger the ground motion, the higher the probability of exceeding some predefined limit state.

For example (following the vertical yellow line), for a 0.2G PGA the probability of exceeding a “Heavy Damage” state is 0.54, the probability of exceeding a moderate damage state is 0.78, and the probability of exceeding a light damage state is almost certain (0.98).

Probabilistic Approaches: Fragility Curves

Reinforced Masonry

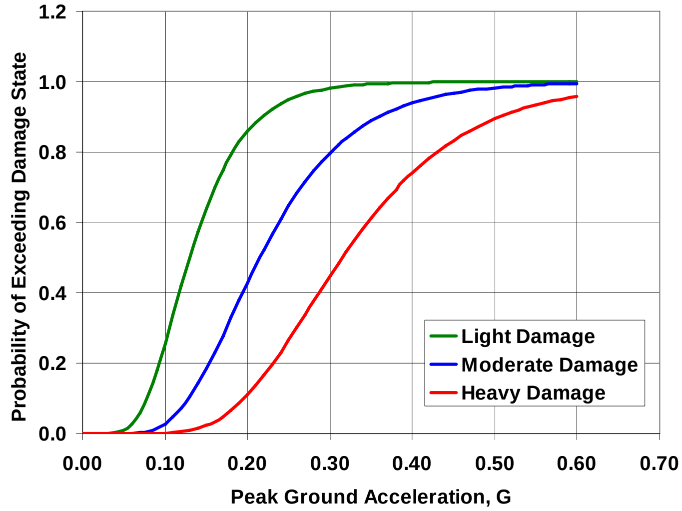


http://www.ceri.memphis.edu/~hwang/

FEMA Topics in Performance-Based Earthquake Engineering *Advanced Analysis 15 – 5c - 71*

Here, a similar set of curves is shown for reinforced masonry (which should perform better than unreinforced masonry).

Probabilistic Approaches: Fragility Curves
Reinforced Concrete



http://www.ceri.memphis.edu/~hwang/

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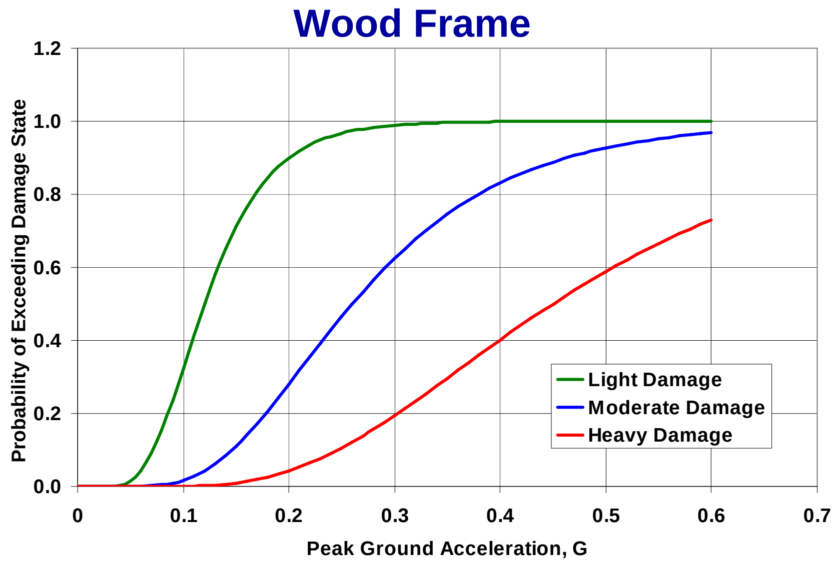
Now, a similar set of fragility curves is shown for reinforced concrete, which should be even better.

Probabilistic Approaches: Fragility Curves



http://www.ceri.memphis.edu/~hwang/

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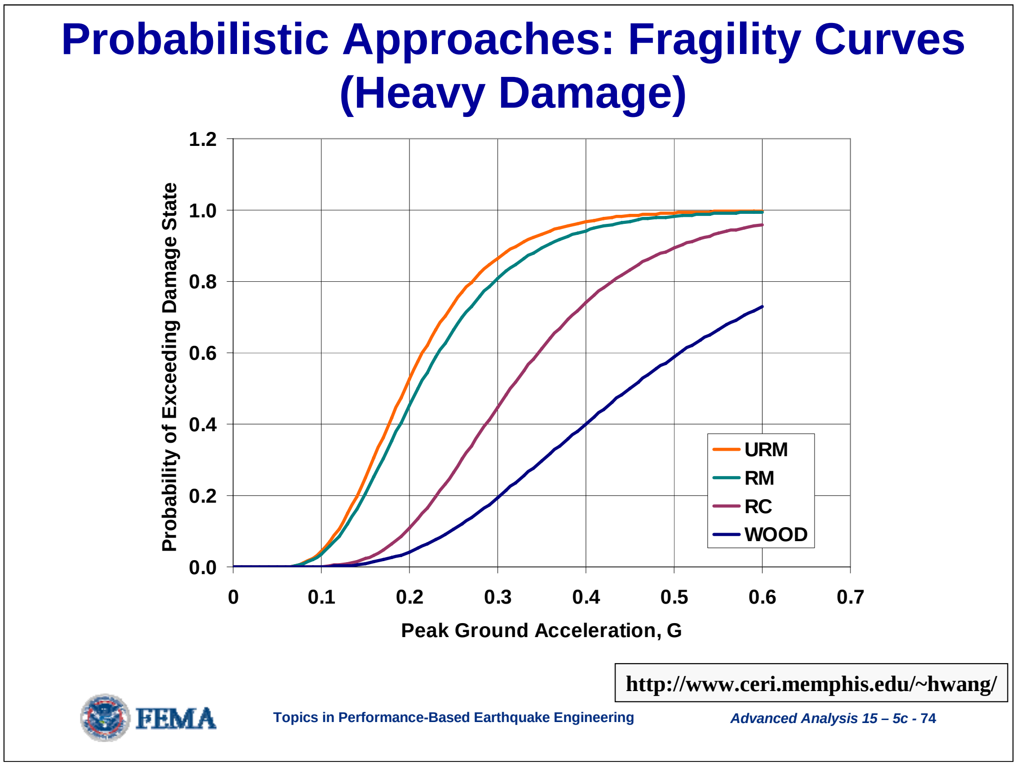
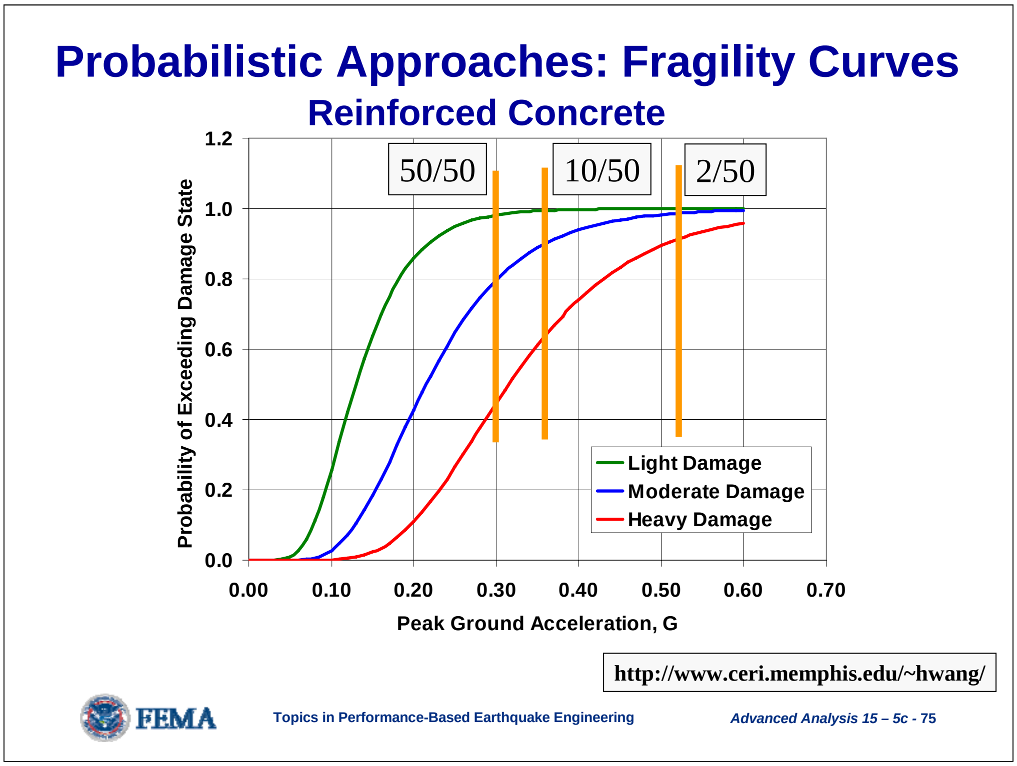
Finally, fragility of wood structures is shown.



This slide compares the four materials. It indicates that URM and RM have very similar probabilities of damage given a ground motion intensity, that RC performs better, and wood is the best. In fact, there is only a 73% probability that the wood structure will be heavily damaged during a 0.6PGA earthquake.



If the levels of probability can be associated with ground motion intensity (as shown) it may be stated that there is, for example, a 90% probability that a reinforced concrete structure will be heavily damaged during ground motion with a 2% probability of being exceeded in a 50 year period.

Where are We Headed with Performance Based Engineering?

- **Performance Basis: Minimize Life Cycle Costs**
  - Realistic Damage Measures
  - Realistic Forecasting of Cost of Repairing Damage
  - Realistic Forecasting of Cost of Loss of Use
- **Analysis Procedures**
  - Incremental Nonlinear Dynamic Response History Analysis
  - Sensitivity Analysis (Deterministic)
  - Probabilistic Assessment of Performance
  - Deaggregation of Probabilistic Results (Deterministic)

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This is a summary. There is clearly a lot to be done before a usable probabilistic framework may be developed for design purposes. PEER is working in that direction, and much attention should be paid to those efforts, as they will likely emerge as the basis for building code provisions of the future.

What We Need

- Ground motion search, scaling, and modification tools for development of suites for nonlinear dynamic analysis
- Reliable damage measures which (hopefully) minimize dispersion in results
- **Rapid** but reliable methods of analysis, including
  - Multiple Ground Motions [7 motions]
  - Incremental Nonlinear Dynamic Analysis [20 increments]
  - Systematic Sensitivity Analysis [10 uncert. X 8 values]
  - Deterministic/Probabilistic Assessment Tools
- Big, **Fast** (Parallel Processing) Computers

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Finally, this is a summary of what is needed. Note that the third bullet requires as many as (7)(20)(10)(8)=11,200 analyses. The more we know about the uncertainties and the effect of those uncertainties on response, the more we can reduce the number of analyses required to obtain a reasonable level of confidence in the results. However, almost any scenario requires bigger faster computers than are currently available in design firms.