How Do Return Periods and Design Provisions for Seismic Loading Compare with Those Used for Other Hazards

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Structural Design Basis

- For a given *Limit State*, load effect shall be less than resistance:
  Q* < *R
- Limit States normally divided into *Safety*, *Serviceability*, and *Durability*
- For most *design situations* loads and resistances are considered *uncertain*, and probability theory is applied to provide *reliable* performance

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Example Design Situation

- Structural element to carry live load, snow load, and wind load: combined load effects to consider (additive):
  - 1.4 D
  - 1.2 D + 1.6 L + 0.5 S + 0.8 W
  - 1.2 D + (1 or 0.5) L + 1.6 S + 0.8 W
  - 1.2 D + (1 or 0.5) L + 0.5 S + 1.6W

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Example Design Situation

- For the same situation, but considering counteracting actions:
  1.6 W + 0.9 D
- Counteracting actions are common where lateral loads are included, and wind also includes upward suction on roofs

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Generalized Load Combination

$$Q = \sum_{i=1}^{n} \gamma_i Q_i^*$$

- γ_i are load factors
- *Q** are reference (nominal) load
- Nominal Snow and Live loads are referenced as the value with a 2% probability of being exceeded in 1 year (the 50 year *MRI*)

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Reliability Basis

$$\sum_{i=1}^{n} \gamma_i Q_i^* \leq \phi R^*$$

- Load factors and resistance factors (γ_i and ϕ_i) are selected to deliver desired level of reliability
- Probability stated thus: P(*Q*-*R*>*0*)
- Level of reliability usually stated as a safety index β, where $\beta = 0$ is *Q* = *R*

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Reliability Basis

- β represents the number of standard deviations away from the mean
- General targets for β:
  - ✓Bridges: 3.5
  - ✓Buildings:

Limit State	Ductile	Brittle
Local	3.0	3.5
Widespread	3.5	4.0

Reliability Basis

- Nominal probabilities of failure represented by values of β:

0	1	2	3	3.5	4
0.5	0.159	0.0228	0.00135	0.000233	3.17E-05

- Reasonable approximation is 0.1% chance of failure in the reference period (50 years for typical structures), thus 2 x 10^{-5} per year (1/50,000 per year)

Reliability Level

- How real are nominal probabilities?
  - ✓Truly rare events not well captured by common statistics; e.g. accidental deaths due to structural failures probably thrown in "all other" category
  - ✓Fire deaths in US in 2003 = 3369, which is approximately 1 out of 10^5 people
- Real values for deaths are probably on the order of one per 10^{-7} per year
  - ✓Less than 1% of nominal reliability

Wind Load

- Reference wind load is not the 50 year MRI load
- W* .=. 500 year wind load / load factor, which is derived from a *wind speed* map showing 500 year MRI wind speeds divided by 1.23
  - ✓Differing wind phenomena lead to differing load factors, an undesirable complication for design purposes

Seismic Loads are Different!

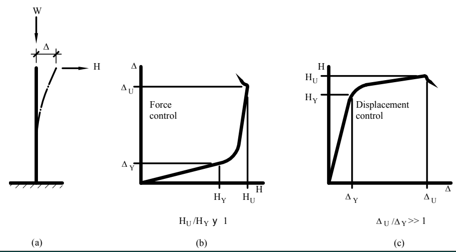
- Reference load is 2500 year MRI, not 50 or 500
- Load factor is 1.0
- Does this mean that the probability of exceeding the limit state is 1/2500 per year?
- Why is it so different?

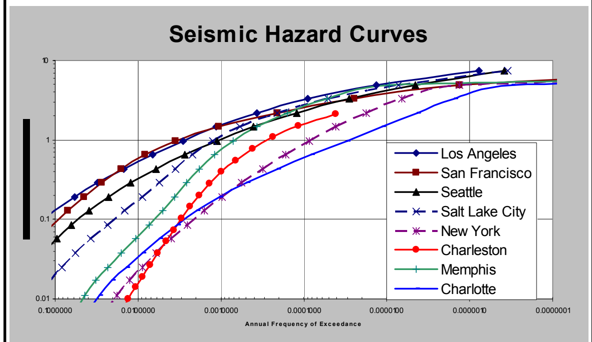
Seismic Loads are Different!

- Limit state for gravity and wind loads is the failure of a member, with varying levels of reliability depending on the nature of the limit state
- Limit state for seismic is collapse of the structure
  - ✓Should this imply a greater level of reliability?
  - ✓Does it?

Displacement and ductility

Margin between yield and ultimate is small for force loads, but can be large for displacement loads





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Questions

- How do you select a load factor?
  - ✓Compare Charleston with Charlotte
- How safe is safe enough?
  - ✓Recall 1/50,000 chance per year of exceeding *member* limit state for wind, snow, and live loads
  - ✓Current thinking is to target 10% chance of collapse given MCE = 1/25,000 /year

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Questions

- Shouldn't the reliability against collapse be greater than against member failure?
  - ✓Member failure under gravity load *can* be close to system collapse under lateral load
- Shouldn't concentration of loss in space and time (for EQ) argue for a greater reliability?
  - ✓Can we afford that?

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Personal Recommendation

- For safety, stay with 2% in 50 year for definition of hazard.
  - ✓Current nominal reliability is not too high
  - ✓Refinements to tune the design value based upon the local slope of the hazard curve could provide methodology to change the nominal exceedence level, but the target reliability should not decrease
- For damage, provide 50 year MRI values

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James Harris

Abstract

The nominal basis for specifying the seismic ground motion hazard for structural engineering design purposes has changed substantially several times over the past half century. The most recent significant change was developed about ten years ago by a joint program of the U. S. Geological Survey and the Building Seismic Safety Council, with support from the Federal Management Agency. It was implemented in the 1997 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, prepared by BSSC. That change has cascaded through the pertinent standards and model building codes, and it is now in use in most of the U.S. As with any change, there has been some controversy. USGS and BSSC are now engaged in a study of how best to specify seismic ground motions for future generations of design standards and building codes. In support of this effort, a systematic comparison of the basis for structural design to resist seismic ground motions over many generations of building codes and of the current basis with that used for structural resistance to other natural hazards has been undertaken. This presentation is a status report on that comparison.

Structural design is based on the concept of providing a minimum level of reliability that predictable limits states will not be exceeded. Limit states are normally divided into safety, serviceability, and durability categories; safety is the primary basis for resistance where seismic hazards are involved. Most structures are potentially subject to a variety of permanent and transient loadings, typically including the action of gravity on self weight, supported contents and activities, and the weights of snow, rain and ice as well as the lateral forces of wind, self-straining actions of various volume changes, and the ground motions from earthquakes. Most of these loads are of uncertain amplitude. The resistance of the structure is also not known deterministically.

The typical limit state equation can be stated thus:

$$\sum_{i=1}^{n} \gamma_i Q_i^* \leq \phi R^*$$

where:

Q_i^* is the effect of one load or action, specified at some reference level

γ_i is the load factor for load i

ϕR^* is the reference resistance

The loads and resistances are defined at nominal levels, and the load and resistance factors are designed to deliver the desired nominal reliability. Where a design situation involves multiple transient loads, several combinations of loads are normally checked to find the controlling design scenario. In each combination, one of the transient loads is assumed to be at its maximum level while all other uncorrelated transient loads are taken to be at their "arbitrary point in time" level. Thus the load factor depends on the position of the load in the combination. With the current basis for specifying reference levels of the loads, some of the most common load combinations where the results of all the permanent and transient loads are additive are:

$$1.2\,D + 1.6\,L + 0.5\,S + 0.8\,W$$
$$1.2\,D + (1.0 \text{ or } 0.5)\,L + 1.6\,S + 0.8\,W$$
$$1.2\,D + (1.0 \text{ or } 0.5)\,L + 0.5\,S + 1.6\,W$$

Where D is the effect of self weight and permanently attached construction, L is the effect of live load (contents and activities), S is the effect of snow, and W is the effect of wind. Notice that the largest load factor rotates among the transient loads. The smaller load factors deliver the arbitrary point in time loads. Where any transient load counteracts the effect of the leading variable, it is omitted from the combinations.

Where the effect of the permanent load counteracts the effect of a transient load, the load combination changes somewhat, because the permanent load becomes part of the resistance. (Common situations include self weight resisting the hydrostatic uplift from flooding or the overturning effect of wind) For wind:

$$0.9\,D + 1.6\,W$$

The load and resistance factors are specified to deliver a nominally small probability that the limit state equation will fail. For working purposes the common reference is to a safety index, β, which is the number of standard deviations that the failure condition is removed from the mean value. The value of β depends on the reference period, the nature of the structural behavior leading to the limit state, and the consequences of exceeding the limit state. For 50 year reference periods, common values of β range from 3 to 4, which correspond to a range in probability of exceeding the limit state in the reference period from 0.00125 to 3.17×10^{-5}. Where the reference loads are specified on the same basis, the larger the uncertainty in the load, the larger the load factor.

If one takes the nominal probability of exceeding a structural safety limit state as 0.0010 and the reference period of 50 years, in a rough sense the corresponding nominal probability of exceeding the limit state in one year is 2×10^{-5}. Comparison with actual statistics for accidental deaths is problematic, because structural failures are rare and a significant portion of the deaths are concentrated in events that occur much less frequently than yearly. In contrast, in 2003, there were 3369 deaths in the U.S. due to fires, which is a rate of about one per 100,000 people. It appears that the long term rate of death due to building structure failures in the U.S. would be approximately two orders of magnitude less than the nominal probability of exceeding a structural limit state.

The reference load for snow and occupancy live loads is the 2% chance of being exceeded in one year, or the 50 year mean recurrence interval (MRI) load. Although the load factor for wind is essentially the same, in the most recent standards the reference load is actually derived from the 500 year MRI wind speed. The derivation is designed to preserve the load factor, in that the wind speed hazard maps are drawn from the 500 year mri wind speed divided by the square root of the load factor (because the load varies as the square of the wind speed). This new approach to specifying the hazard is taken because the statistics of extreme wind speeds is different in regions prone to hurricane as compared to other regions, thus the new approach delivers a more consistent risk.

The basis for the seismic ground motion hazard is quite different. The basic motion is specified based upon a 2% chance of being exceeded in 50 years, or a 2500 year MRI value, and downward adjustments are made relatively close to sources with frequent activity. The adjustment applies in areas where the uncertainty in ground motion attenuation predictions tends to drive the computation of the ground motion, and it may introduce an element of consistency in the risk. The load factor on the seismic action is taken as 1.0. The load factor of 1.0 does not imply the limit state is reached when the actual ground motion reaches the 2500 year MRI value, however. The design equation still includes the factor on the nominal resistance, and the current thought about the nominal probability of collapse is that there is a 10% chance of collapse given the occurrence of the 2500 year MRI motion. This gives a nominal probability of exceeding the limit state for seismic loads that is similar to the nominal probabilities for exceeding limits states driven by other hazards.

However, there are other significant differences between the design criteria for seismic and wind. Like other ordinary loads, the limit state for wind design is exceeding the strength in one structural member. For seismic design, the limit state is structural collapse. A significant advantage is taken for ductility because the load is a displacement based action, in contrast to gravity loads and wind, which are force based actions. One could argue that the difference between the limit states would call for greater reliability in the seismic case.

The reason that the seismic load is specified based upon a very long reference period is similar to that for wind: the slope of the hazard curve varies dramatically from point to point. However the slopes of typical seismic hazard curves are very different than those for wind: if the seismic load were specified as a 50 year MRI load, the load factors necessary to reach the 2500 year MRI level would range from around 4 to perhaps as much as 50. The next generation of improvement in specifying seismic ground motions may be to adjust the probabilistically computed values for the slope of the overall seismic hazard curve in a risk-consistent fashion. There is no apparent reason to lower the target level of reliability.