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MASONRY DESIGN IN HIGH SEISMIC AREAS

GARY C. HART[1]

ABSTRACT

The design of masonry structures in high seismic areas requires a special design approach that directly incorporates strength and ductility considerations. This paper addresses both of these as well as the description of the earthquake ground motion.

INTRODUCTION

It was with considerable honor that the author accepted the invitation from Professor Robert Drysdale. to present a keynote lecture at the 7th Canadian Masonry Symposium. The honor of being one of three keynote speakers together with Professor Luigia Binda from Milan, Italy and Professor Gary T. Suter from Carleton, Canada is only out weighed by the responsibility for meeting the expectations of Professor Drysdale and the Symposium Organizing Committee.

The Canadian Masonry Symposium occurs every three years and it always brings together architects, building officials, building science specialists, contractors, educators and structural designers. The conference has always provided the author with an opportunity to learn from others and also to clarify some pre-conference ideas that needed the intellectual stimulus that this conference provides. Therefore, in this spirit this keynote lecture seeks to present the author's current ideas on Seismic Design of Buildings in high Seismic Areas. These ideas will first focus on the basic philosophical framework that the author believes is essential for the future of seismic design. Then the ideas will focus in greater detail on specific structural concepts that must be included in a seismic design criteria.

[1] Professor, University of California, Los Angeles, and Senior Consultant, Hart Consultant Group, Los Angeles, CA, USA 90024

The structural engineering community (university, researchers, structural engineers and building officials) is faced with many pressures from many different individuals and groups as a result of recent earthquake damage. Two very strong forces fueled by the 1994 Northridge and the 1995 Kobe earthquakes, are irreversible and will change the structural engineering design in high seismic areas. One force is the unprecedented increase in the importance of understanding real ground motion shaking and real structural performance to this shaking. The second force is the demand for greater quality control in construction and design documents.

SEISMIC DESIGN PHILOSOPHY

Seismic design in high seismic regions must be founded on the following three cornerstones:

(1) Limit State Design,

(2) Structural Reliability, and

(3) Design Deviation Barriers.

Each of these three cornerstones will now be discussed,

LIMIT STATE DESIGN

Structural engineers do not like to talk about failure. However, a rational design criteria must have a definition of failure. Limit State Design is based on the need to define and quantify the possible modes of structural engineering components of system failure and at the same time, to the maximum extent possible, avoid the use of the word failure by defining the term Limit State.

To illustrate the essence of Limit State Design consider the wall load deflection curve shown in Figure 1. For communications purposes let us focus on the Limit State 2 which corresponds to the start of yielding of the steel in the wall. Limit State 2 exists if and only if the strain in the steel is exactly equal to the yield strain of the steel.

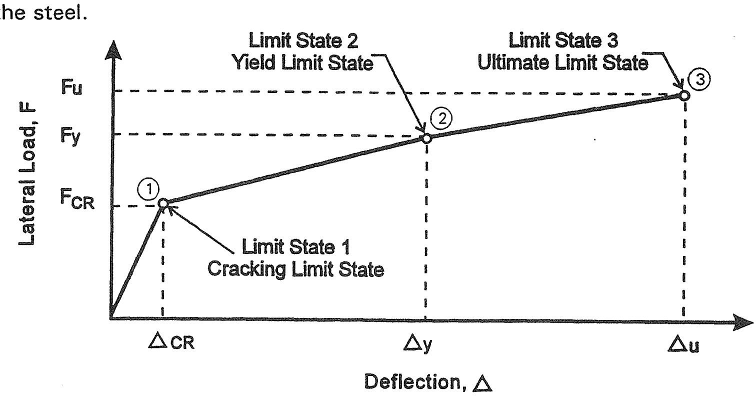


FIGURE 1 WALL LOAD VS. DEFLECTION CURVE

The development of a design criteria requires that three questions be answered for Limit State 2 or, in fact, for any limit state. First, do we care about the limit state and should the design criteria address it at all? Second, what are the consequences of reaching this limit state and how much should our safety cushion, or safety level be to protect us from the limit state occurring? Third, how often are we willing to accept the occurrence of this limit state? To illustrate how these questions are now addressed in one city in a high seismic area consider the city of Los Angeles high rise building design criteria.

First, the structural engineers who developed the City of Los Angeles high rise buildings criteria did care about the limit state corresponding to the first yield of the steel. They have identified it as a design consideration that must be addressed by the structural engineer who designs a building and, in particular, he or she must calculate the nominal yield moment capacity of the structural component (e.g. referred to as M_n in this discussion). Second, the consequences of having the load induced moment equal the limit state and then exceed it are structural damage. Thus, this is a Structural Damage Limit State and the existence of the limit state will have an impact on the building's function and will also have an economic impact. Third, the answer to how often the decision maker will accept the occurrence of the limit state can be inferred from the earthquake selected for the Design Level Earthquake. In the city of Los Angeles building design criteria the Design Level earthquake that has been selected is an earthquake with a 50% chance of being exceeded in the next 50 years. This earthquake can be expected to occur on the average once every 72 years. Thus, in this case the City of Los Angeles is willing to accept the occurrence of the first yield limit state once every 72 years.

STRUCTURAL RELIABILITY THEORY

It is fantasy to think that we are able to calculate an exact value for the probability of failure for a limit state and not to be laughed at by good structural engineers. In fact, the very attempt to present a position that the calculation of the probability of failure is exact undermines the introduction of the principles of structural reliability into the structural engineering design process. What is clear is that we need to have a rational way to calculate the approximate safety associated with the occurrence of a limit state. We also need to have a way to reward better than average quality control in the construction process. Furthermore, we need to be able to reward the extra effort associated without he use of new analytical modeling technology when it clearly results in increased confidence in our final design. The principles of structural reliability theory can and will accomplish all of the above needs.

To illustrate the role of structural reliability theory consider Limit State 2 as previously discussed which corresponds to the start of yielding of the steel. The issue of how do we quantify safety is not new. For example, in the United States the ICBO Evaluation Services, Inc., must address this issue in evaluating new products and it has used the following basic approach to establish acceptable factors of safety. A proponent of a new system first calculates the load corresponding to a limit state based on the design equations in the proposed design criteria. Then, the proponent conducts an experimental test and measures the load that corresponds to the limit state. If the ratio of the experimental load to the calculated load equals or exceeds 2.5 for a working stress design criteria or 1.7 for a strength design criteria then an acceptable level of Safety has been attained.

In structural reliability theory the role of the safety factor is played by a term called a SAFETY INDEX. Acceptable values of the safety index have been discussed and representative values for different types of limit states are 1.5 to 2 for serviceability (or not "serious") limit states and 3 to 4.5 for ultimate (or "serious") limit states. The more undesirable the consequences of the existence of the limit state the greater the minimum acceptable value of the safety index. The author and Professor Drysdale presented this concept of Safety Index to the ICBO Evaluation Service, Inc., for approval of concrete and masonry walls and they adopted it as an alternative to the ratio based method discussed in the previous paragraph.

Structural reliability theory also enables us to work within the existing familiar bounds of structural design calculations to reward good quality control and technology transfer. It does this through the rational determination of a "phi factor" or capacity reduction factor. Consider for example the yield moment limit state where the structural engineering equation in the design criteria is

$$M_d = \Phi M_n \qquad (1)$$

where

M_n = Nominal value of the yield moment capacity calculated by the designer using the specifications in the design criteria.

Φ = Capacity reduction factor for this limit state which is the first yield moment.

M_d = Design value of the yield moment capacity that is compared with the load induced moment demand on the structural component.

It can be shown that one way to calculate a value for the capacity reduction factor is to use the equation

$$\Phi = (M_e/M_n)\exp[-075\beta V_m] \qquad (2)$$

where

M_e = Expected value of the yield moment capacity.

V_m = Coefficient of variation of the yield moment capacity which reflects the uncertainty in the field moment as we calculate it using M_n compared to what is actually constructed.

β = Safety Index.

In high seismic regions masonry structural design criteria must follow an approach called EXPECTED VALUE LIMIT STATE DESIGN. This approach is simple in concept. All it requires is that the structural engineering design criteria always seek to propose equations and approaches for structural engineers to use that results in the best possible estimates of what the real value of the calculated quantity is going to be and not what is typically done now in design use "lower bound" estimates. The goal is to address reality. In the context of Equations (1) and (2) this means that in the Expected Value Limit State Design approach the nominal design value, M_n should be as close as possible to the expected design value, M_e, using the Expected Value Limit State Design approach is follows that M_n is approximately equal to M_e and Equation (2) becomes

$$\Phi = \exp[-075\beta V_m] \qquad (3)$$

Masonry design criteria in high seismic zones must encourage and reward quality control and technology transfer. The value of the capacity reduction factor increases as our confidence in estimating the value of the response quantities used to define the limit state increases. Thus, the masonry design criteria must directly account for two or three different levels of quality control with their relative uncertainty levels. It must reward the better and typically more expensive quality control levels with a higher value of the capacity reduction factor and a decrease in the quantity of material required. Similarly, the use of analytical models such as finite element models that clearly provide more accurate estimates of structural performance can and must be rewarded with higher values of the capacity reduction factor.

DESIGN DEVIATION BARRIERS

Design criteria must, for previously noted reasons, have structural engineering design equations that can be easily understood and readily used by structural engineers. However, it is essential that the design criteria in high seismic regions where very sophisticated nonlinear dynamic computer models are essential to understand performance must promote and reward innovation and technology transfer. The dilemma of how to rationally deal with these two seemingly conflicting needs was solved by the author as part of his work on the 1991 UBC Base Isolation of Building Design Criteria.

The 1991 UBC Base Isolation Design Criteria contains a static design procedure and associated set of assumptions and equations that can be used by most structural engineers. The use of the static design procedure requires a minimal understanding of structural dynamics, earthquake engineering or structural analysis. In addition, the static design procedure meets the basic needs of structural engineers who are performing review calculations and provides important assistance in the preliminary design phase of a project.

Now, a key element of the new base isolation design criteria that the author proposed and was successful in having in the criteria was that if the structural engineer so chooses he or she may use a more sophisticated design approach where a nonlinear time history analysis is used for the structure above the isolation and typically saves construction cost because loads less than static design loads can be used for design. Therefore, technology transfer becomes cost effective! However, another key element is that no matter how many calculations are performed and no matter how sophisticated the calculations are the final design cannot result in a design below a specified minimum acceptable value. Thus, there exists a "safety net" in the design criteria and it is this safety net that the author calls a Design Deviation Barrier.

DESIGN/EARTHQUAKE GROUND MOTION

The design of the masonry structures in high seismic areas contains two basic components. One is the description of the ground motion and the second is the description of structural performance. This section of the paper addresses the ground motion.

The DESIGN LIFE OF A BUILDING is its expected duration of useful existence, i.e. its "life." The term can be misleading since very few buildings are demolished at the end of their design life. Many buildings are demolished before their design life is reached and many other buildings remain long after the design life has expired. However, the engineer and the building owner (or city Officials) in high seismic areas must decide how long the structure will be in service (and thus exposed to earthquakes) so that a rational probabilistic assessment of earthquake risk can be accomplished. This decision is made by considering various factors such as the sociological or personal needs of the owner or community. Typically, most buildings are assigned a design life of fifty years. Thus, in assessing earthquake risk, the probability of an event occurring is usually expressed over a fifty year duration.

Earthquake ground motion can be described in terms of its intensity, frequency content, and duration of shaking. Ideally, any method of representing an earthquake ground motion for use in design should incorporate these three descriptive variables. An ensemble or set of acceleration time histories that represent the ground shaking expected at a building site is the best way to describe the ground motion at a site. However, time history analyses can be computationally demanding and thus time history ensembles are not the most common way of describing ground motion. A smoothed elastic response spectrum for a single degree of freedom oscillator is typically used to characterize ground motion. Response spectra quantify the intensity of the frequency content of the ground motion but do not directly provide any information of the duration of the shaking.

Smoothed response spectra reflect the intensity of ground motion expected at a building site by using normalizing parameters. Several potential parameters exist but for this paper the parameter selected is the value of the 5% damped elastic response spectrum at a period of 1.0 sec. This is hereafter denoted S_A. Using this parameter earthquake ground motion intensity can be defined using probability principles as is done with design loads for other hazards such as wind and floods.

The risk of particular level of ground shaking occurring can be expressed as the probability of occurrence in a given period of time, usually the design life of the building. Another way of expressing risk is to state the Mean Return Period (or Mean Recurrence Interval or simply the Return Period as it is usually called) of the given earthquake. The Return Period of a specific level of ground motion represents the expected average time between earthquakes with equal or greater intensity. The term can be misleading since it does not mean that the earthquake will occur every Return Period.

A relationship between probability of exceedence over a given period of time and Return Period can be developed. For illustrative purposes, earthquake intensity can be defined in terms of the random variable S_A. If the Return Period for an earthquake with an S_A value is:

$$P(S_A \geq a)_{1yr} = \frac{1}{T} \tag{4}$$

It follows that the probability of not being exceeded (or probability of non-exceedence) in any single year is:

$$P(S_A < a)_{1yr} = 1 - \frac{1}{T} \tag{5}$$

The probability of not being exceeded during a period on N years is the product of the probability of non-exceedence during each of the N years and the probability of exceedence within a period of N years is:

$$P(S_A < a)_{Nyr} = 1 - \left(1 - \frac{1}{T}\right)^N \tag{6}$$

This equation can be rearranged so the Return Period is expressed as:

$$T = \frac{1}{1 - \left[1 - P(S_A > a)\right]^{1/N}} \tag{7}$$

Thus, from Equation (7), an earthquake termed the Design Basis Earthquake (DBE) with a 10% probability of being exceeded in 50 years has a Return Period of:

$$T = \frac{1}{1 - (1 - 0.1)^{\frac{1}{50}}} = 475 \text{ years} \tag{8}$$

And the Return Period for an earthquake defined by the 1991 UBC as the Maximum Credible Earthquake (MCE) with a 10% chance of exceedence in 250 years is:

$$T = \frac{1}{1 - (1 - 0.1)^{\frac{1}{250}}} = 2373 \text{ years} \tag{9}$$

Also, the probability of a Maximum Credible Earthquake occurring during a 50 year interval is obtained from Equation (6) to be:

$$P(S_A \geq a)_{50yr} = 1-\left(1-\frac{1}{2373}\right)^{50} = 2.1\% \qquad (10)$$

This means that in a fifty year time period, the DBE is about five times as likely to occur as the MCE.

The first step in a statistical characterization of ground motion is to select a descriptive parameter. The maximum S_A in the design life of a masonry structure is chosen as the illustrative descriptive variable in this paper. The next step is to assume a probability density function (PDF) for the maximum value of the S_A to occur in the Design Life of the building. A PDF is a mathematical function usually with two parameters. Thus, when these two parameters are assigned a value the PDF is defined.

The maximum S_A in a 50 year time period is now defined as a random variable of interest. Image for illustrative purposes that for a particular building site the S_A for the DBE with a 10% probability of being exceeded in fifty years is 0.4g. Similarly, imagine that the value of S_A for the MCE which has a 2.1% probability of being exceeded in fifty years is 0.53g. These two pieces of information, i.e. S_A values at DBE and MCE probability levels, will be sufficient to define the two parameters for the PDF.

The Type II probability density function is a probability density function commonly used earthquake risk assessment for the parameter S_A. The density function is given by:

$$p(S_A) = \left(\frac{k}{u}\right)\left(\frac{u}{S_A}\right)^{k+1} \exp\left[-\left(\frac{u}{S_A}\right)^{k}\right] S_A \geq 0; u,k > 0 \qquad (11)$$

The cumulative distribution of this function or probability distribution function is:

$$P(S_A) = \exp\left[-\left(\frac{S_A}{u}\right)^{-k}\right] \qquad (12)$$

or alternatively

$$u = S_A\left[\mathrm{In}\left(\frac{1}{P(S_A)}\right)\right]^{1/k} \qquad (13)$$

In a fifty year period, the two sets of values for S_A and $P(S_A)$ have been provided for the DBE and MCE as previously noted and they are:

S_A	=	0.40g,	$P(S_A) = 0.90$	(Design Basis Earthquake)
S_A	=	0.53g,	$P(S_A) = 0.98$	(Maximum Credible Earthquake)

Substituting the above values into Equation (13) yields two equations with two unknown coefficients u and k. Solution of these equations results in:

$$u = 0.273$$
$$k = 5.899$$

These values u and k fit the Type II probability density function in Equation (11) to the known seismic risk. Figure 2 shows a plot of the PDF of S_A for this example. The PDF definition of ground motion is resisted by many scientists and engineers because they wish to provide single value for the Maximum ground acceleration. The design of masonry structures in high seismic areas must adopt the PDF approach and recognize that there is always a finite probability of failure. It is only when we adopt the PDF approach can we have a rational basis for discussing and defining earthquake ground motion.

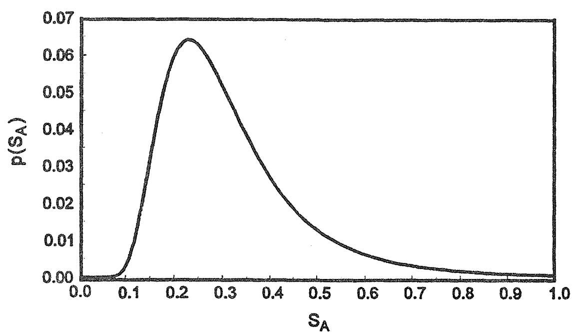


FIGURE 2 PROBABILITY DENSITY FUNCTION OF MAXIMUM SPECTRAL ACCELERATION AT 1.0 SEC PERIOD IN 50 YEARS (S_A)

DESIGN / STRUCTURAL PERFORMANCE

General

The quantification of the possible structural performance of masonry components and systems is absolutely essential in high seismic areas. As an introduction to how this is done and the role that limits states play in this step consider a masonry prism. A typical masonry prism is 16 inches tall and is constructed by placing two masonry units in a stacked bond. A masonry prism test measures the axial compression load/deflection behavior of this prism. An important part of this measurement is the maximum compressive load that can be carried by the prism. The maximum compressive stress for the prism is obtained by dividing the maximum compressive load by the net area of the prism. The strain in the prism is the deflection divided by the height of the prism. Figure 3 shows a general masonry stress versus strain curve for a prism. Table 1 defines the behavior and limit states shown in this figure. Figure 4 shows an actual stress versus strain curve.

These figures and table clearly show how it is possible to present structural performance in terms of limit states and behavior states. The former being equations for use in mathematical formulations and the latter being word descriptions for use in communications with users and clients.

Ductility

It is our desire in high seismic areas to have masonry structures be designed to be capable of considerable inelastic behavior without a failure. This we call ductile design.

Dr. Nigel Priestly conducted compressive loading tests in New Zealand on masonry prisms with a type of confinement comprised of thin galvanized plates placed in the bed joints of the masonry. Experimental research conducted at the University Colorado that was funded with a U.S. Government Small Business Innovative Research Award under the direction of Dr. Hart, Dr. James Noland and Dr. Robert Englekirk showed that other methods of confinement could also produce an enhancement of the post maximum stress load/deflection performance. This type of confinement resembles a comb in form and is called a SEISMIC COMB and is manufactured by the DUR-O-WALL corporation. The plate and the comb provide a Compressive Confinement to the grout and masonry unit around the vertical reinforcing steel bar and tend to hold the rebar/grout /masonry unit together and significantly improve the post peak stress performance.

To illustrate how the performance of a prism in compression can be used to understand wall performance consider the wall shown in Figure 5. Figure 5 shows a schematic of a section of the wall. Prior to the application of any lateral load the wall section is typically in a state of uniform compression due to the vertical dead and live loads acting on the wall. With the application of any lateral load, the strain now varies in a linear manner over the length of the wall. Figure 5 shows the linear strain variation. It also shows the compressive strain at the end of the wall which

TABLE 1 BEHAVIOR AND LIMIT STATES FOR MASONRY PRISMS

State	Description
Behavior State 1	In this behavior state prisms under compressive loading exhibit no significant signs of physical distress.
Limit State 1	Limit State 1 exists when $e_m = e_c$. This is the prism's serviceability limit state.
Behavior State 2	The compression strain e_m exceeds e_c and the stress has not reached the maximum value.
Limit State 2	Limit State 2 exists when $e_m = e_u$. Alternatively stated as $f_m = f'_m$.
Behavior State 3	The compressive strain e_m exceeds e_u and the stress decreases in value from its maximum value.
Limit State 3	This limit state exists when the stress in the prism has been reduced by 50% from its maximum value. This is written as $e_m = e_{mu}$. This is called the Maximum Usable Strain.
Behavior State 4	The prism experiences a strain e_m greater than e_{mu} and even though the prism has exhibited significant physical distress it is capable of carrying a compressive load which is equal to or greater than 20% of f'_m.
Limit State 4	This is the limit state corresponding to the end of Behavior State 4. This limit state exists when $f_m = 0.2 f'_m$.

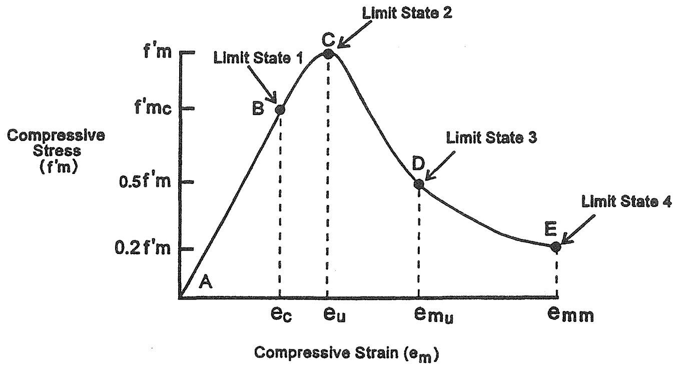


FIGURE 3 CONFINED CONCRETE MASONRY STRESS VS. STRAIN BEHAVIOR

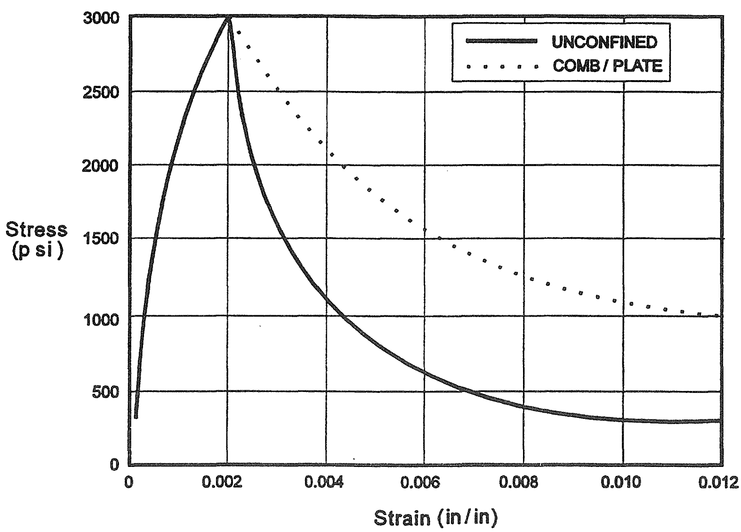
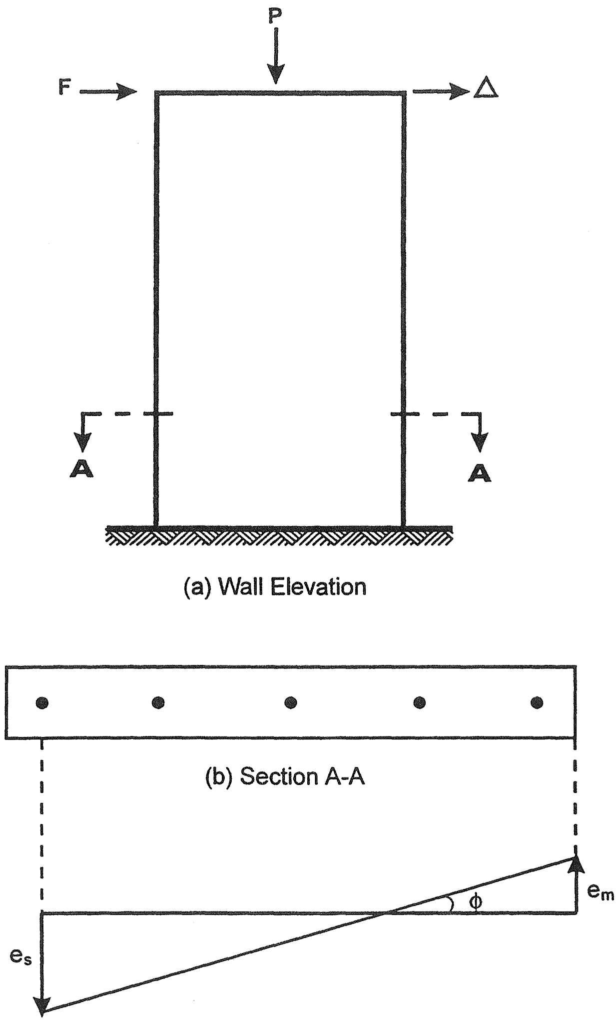


FIGURE 4 PRISM STRESS-STRAIN CURVE



(c) Linear Strain Variation at Section A-A

FIGURE 5 WALL AND SECTION A-A

is denoted as e_m. Figure 5 shows the tensile strain in the vertical steel bar located furthest from the neutral axis and this tensile steel strain is denoted as e_s.

The performance of this one section of a wall can be described in terms of either the moment imposed on this section of the wall or, alternately as the value of the maximum compressive strain in the masonry which occurs at its extreme compressive fiber. Imagine a small but finite moment applied to this section of the wall. The moment will induce a rotation of the cross-section and a tensile strain e_s in the steel and a compressive strain e_m in the masonry. The exact value of these strains is calculated using equilibrium conditions. As the lateral load increases the moment on the section increases until the masonry at the section at its extreme tension fiber experiences a tension stress equal to the modulus of rupture of the masonry. This moment is called the Cracking Moment.

When the lateral load induced moment on the section is equal to the cracking moment it is called the CRACKING LIMIT STATE and is denoted as Limit State 1. The wall at this section is said to be in Behavior State 1 if the lateral load induced moment is greater than zero but less than cracking moment. If the lateral load is increased beyond the load that corresponds to Limit State 1 then the section is in Behavior State 2. When this lateral load produced a moment that causes the extreme reinforcing bar to reach its yield stress, or alternately, yield strain, then the section is at Limit State 2. This limit state is called the YIELD LIMIT STATE and it corresponds to first yield of the tension steel. The moment in this situation is called the YIELD LIMIT STATE and it corresponds to first yield of the tension steel. The moment in this situation is called the Yield Moment, M_y. If the lateral load is now increased further the section will experience structural damage because the steel that is strained beyond the yield stain will experience plastic deformation. This is Behavior State 3. The increase in lateral load will eventually produce a compressive strain in the masonry that is the maximum usable strain. The maximum usable strain for masonry without any confining steel is 0.003. If methods of confinement, such as the DUR-O-WALL Confinement Comb, are used then the maximum usable strain is 0.006 or greater. When the masonry strain in compression reaches the maximum corresponding to this lateral load it is called the Ultimate Moment, M_u, and Limit State 3 is called the ULTIMATE LIMIT STATE. The curvature of a cross-section of a wall at a given moment is defined to be the load induced strain at the extreme masonry compression fiber divided by the distance from the neutral axis to this extreme masonry compression fiber.

A Moment-Curvature plot can be developed at any wall section and these moment curavature plots can be used to develop plots of the wall's load versus deflection. Figure 6 shows a plot of the load versus deflection for a wall with and without conginement steel.

Ductility is increased with the increase in the maximum uable strain as shown in Figure 6. Therefore, confinement of vertical steel in compression is essential when the strains in the masonry exceed approximately 0.002.

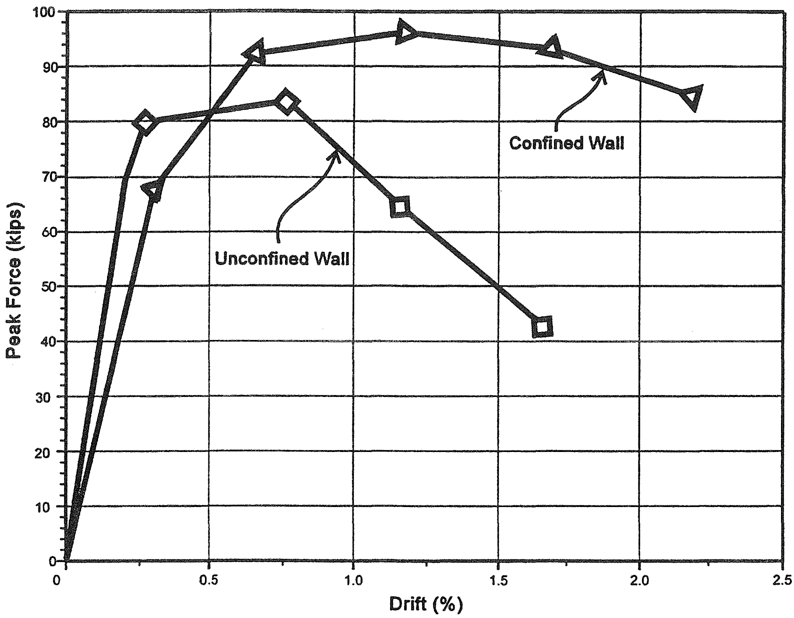


FIGURE 6 WALL LOAD VS. DEFLECTION CURVE

Ductility is also directly related to the quantity of vertical tension steel in the wall. It is desirable to have moderate amounts of steel to promote ductile behavior. In beams that do not have significant axial loads limiting the steel can be done in a straightforward way with a calculation of the strain in the steel at the ultimate limit state. Strains in the steel of 3 to 5 times the steel strain promote excellent ductile behavior.

The axial load on a wall is a very important design parameter when ductility is needed. Figure 7 shows a plot of the moment capacity versus axial load. It is very clear from this figure that when axial loads exceed 20% of the area times the maximum compressive strength ductility is very limited.

Ductility and strength are often at odds. This is because as we increase the axial loads or quantities of steel we increase the strength, e.g. yield moment, of the wall and at the same time we decrease the ductility of the wall. Therefore, we require more load to yield the wall but when it yields it absorbs very little extra energy before failure because of its limited ductility.

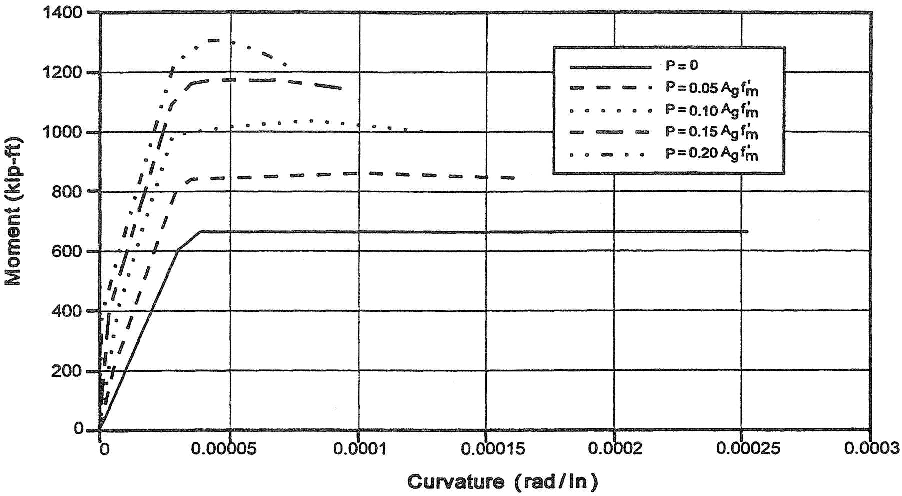


FIGURE 7 WALL MOMENT PERFORMANCE VS. AXIAL LOAD

Distribution of Tension Steel

The uniform distribution of the vertical steel in piers and walls and the horizontal longitudinal steel in beams is strongly recommended in high seismic areas. Figure 8 shows a cross section of a reinforced concrete masonry beam with a uniform distribution of the horizontal steel. One steel reinforcing bar is placed in each course of concrete masonry. The most common alternative to this uniform distribution of steel is to use only 2 bars and place of one bar in the top and one in the bottom masonry units. This is referred to as concentrated steel.

The 8 inch thick and four unit deep masonry beam shown in Figure 8 has one #6 bar located in each masonry bond beam unit. Typically, masonry beam members do not have confinement steel and, therefore, it is reasonable for this beam example to use a value of 0.003 for the maximum compressive strain in the concrete masonry. If we assume a maximum masonry compressive strength of 2,000 psi then the ultimate moment capacity of the beam is 122 kip - feet. Figure 9 shows the cross section of the beam in the state of maximum compressive strain. The neutral axis, as shown in Figure 9 extends a distance of c inches from the compression face of the beam when the maximum compressive strain is 0.003.

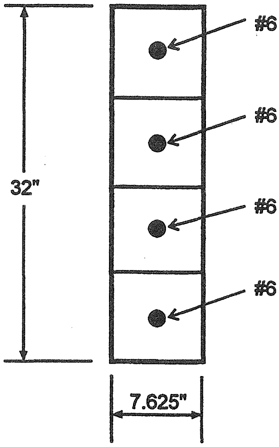


FIGURE 8 UNIFORM DISTRIBUTION OF STEEL

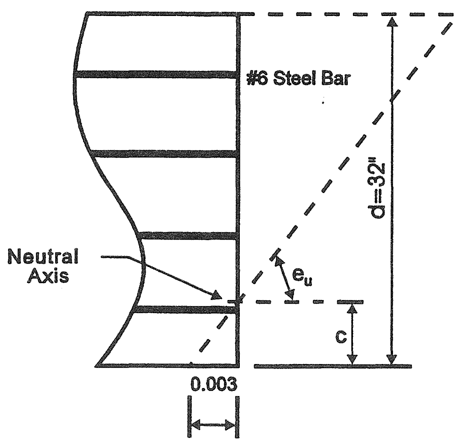


FIGURE 9 STRAIN PROFILE AT ULTIMATE MOMENT

Figure 10 shows how the moment capacity of the beam varies as a function of the compressive strain in the masonry at the extreme compression fiber. At the moment that the beam is subjected to increases so does the strain in the extreme compression fiber. Beam equilibrium calculations can be used to relate the moment and strain for all strains up to 0.003. The numbers 1, 2 and 3 correspond to the state of extreme fiber compressive strain when one more of the reinforcing bars yield. Figure 10 also shows how the moment capacity changes for a beam wit concentrated steel and one #8 bar in the top and bottom units.

Note that there is a constant increase in the moment capacity of the beam as the steel bars sequentially yield. The moment corresponding to Point 1 is the moment capacity of the beam when the first reinforcing bar yields. This is a yield limit state for the beam and the yield moment is equal to 86 kip-feet. After the first bar yields in the concentrated steel situation or after the third bar yields in the distributed steel situation there is little change in the moment capacity of the beam. The moment capacity of the beam is equal to 112 kip-feet when the ultimate limit state occurs at a strain of 0.003. The ratio of the ultimate moment to the yield moment is (112/86) or 1.3.

A plot of the location of the neutral axis versus the strain at the extreme compression fiber can be developed. Figure 11 shows the decrease in the size of the compression zone as the compressive strain increases. The compression zone at the yield limit state is 22% of the depth of the beam. The depth of the compression zone decreases to 16% at the ultimate limit state.

The slope of the moment versus strain plot can be directly related to the stiffness of the beam and it is clear from Figure 10 that there is a sharp decrease in stiffness when the single reinforcing bar yields. This performance is in sharp contrast to the distributed steel design where the stiffness experiences a gradual decrease with strain as the reinforcing bars successively yield. In high seismic areas it is critical that this rapid change in stiffness be evaluated for the specific project because it may mean a significant redistribution in load.

Figure 11 shows that the location of the neutral axis changes very rapidly after the single reinforcing bar yields. The depth of the compression zone decrease to 12% of the depth of the beam or 3.8 inches at the ultimate limit state. The distributed steel design has a more gradual decrease in the depth of the compression zone with strain increasing and significantly larger compression zone at the ultimate limit state. The larger compression zone for the ultimate limit state provides a larger area for the shear forces to act on the beam and thus is strongly recommended in high seismic areas.

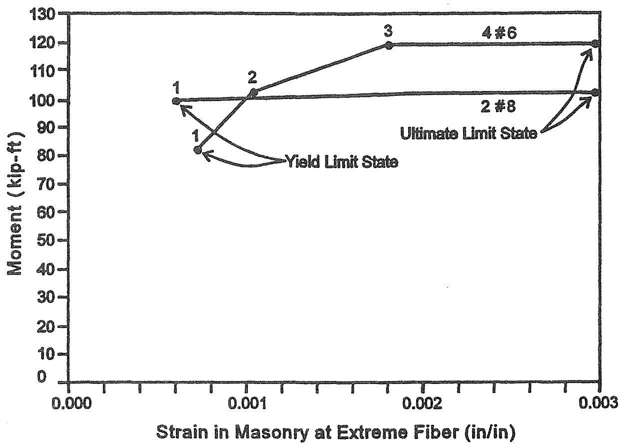


FIGURE 10 MOMENT CAPACITY VS. COMPRESSIVE STRAIN

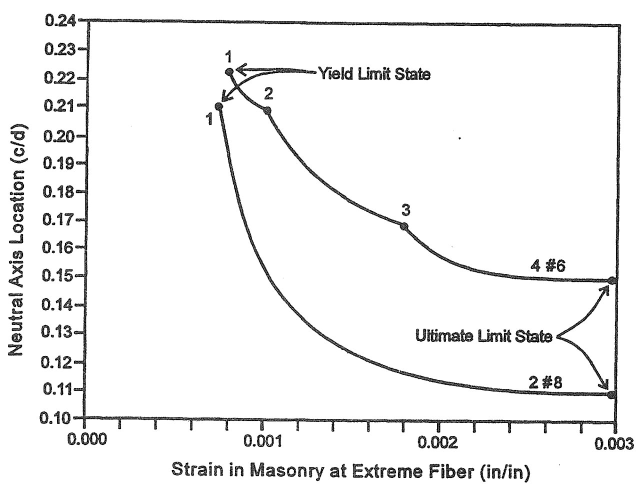


FIGURE 11 NEUTRAL AXIS LOCATION VS. COMPRESSIVE STRAIN

Performance Definition Using Limit States

It is essential in high seismic regions to understand how the performance of the structure changes as the forces and deformation increase in magnitude. To illustrate this consider a cantilever beam subjected to an concentrated load.

The moment that must be applied to the beam in order to cause the beam to crack at the support is called the **Cracking Moment.**

$$M_{cr} = \text{cracking moment} = Sf_r$$

where

S = section modulus.
f_r = modulus of rupture.

It follows that the Cracking Moment Limit State is

$$S = (1/6)\ 7.624\ (40)^2 = 2033 \text{ in}^3$$

$$f_r = 4\sqrt{f'_m} = 155 \text{ psi}$$

$$M_{cr} = 2033\ (155) = 315 \text{ kip-in} = 26.3 \text{ kip-ft}$$

The strain in the extreme masonry tension fiber at the start of cracking is

$$e_m = \frac{f_r}{E_m}$$

where

e_m = masonry strain.
E_m = Modulus of Elasticity of Masonry
$= 750\ f'_m$
$= 1{,}125{,}000$ psi $= 1{,}125$ ksi

Therefore, since the section is uncracked the maximum compression and tension strain in the masonry is

$$e_m = \frac{155}{1{,}125{,}000} = 1.38 \times 10^{-4} \text{ in/in}$$

The neutral axis for the uncracked beam section is at the center of the beam. Therefore, the curvature of the beam cross section, which is defined to be the rotation of the cross-section is

ϕ_{cr} = curvature of beam cross-section at the cracking limit state where c = distance from neutral axis to extreme compression fiber.

It follows that

$$\phi_{cr} = \frac{1.38x10^{-4}}{(40/2)} = 6.89 \times 10^{-6}$$

It is preferred by many engineers to refer to this as the *tensile cracking strain limit state*. It is very important to note that the moment capacity for a limit state can be related to the strain in the masonry.

$$e_{d,EQ} \leq \phi_{fr}\, e_{fr} \tag{14}$$

where $e_{d,EQ}$ = Demand strain on the beam at the user selected return period.
ϕ_{fr} = Capacity reduction factor for cracking limit state

The moment that must be applied to the beam in order to cause the steel in tension to reach its yield stress and yield strain is the yield moment limit state.

The strain in the tension steel at the yield moment is known for this limit state and is equal to the yield strain of the steel. The steel reinforcing bar in tension is at its yield stress and strain at the yield moment and thus

e_y = steel yield strain.
= yield stress/Modulus of Elasticity.
= 60,000 psi / 29,000,000 psi = 0.00207

The design equation for the yield limit state and, for example, the client / user defined MPE ground motion, is

$$M_{d,MPE} \leq \phi_y\, M_y \tag{15}$$

$M_{d,MPE}$ = Demand moment on the beam at the Maximum Probable Earthquake ground motion.

M_y = Expected yield moment capacity.
ϕ_y = Capacity reduction factor for yield moment.

As an alternative consider the design equation in terms of strain. The design equation for the maximum usable strain limit state and, for example, the client/user defined DBE ground motion is

$$e_{d,DBE} \leq e_{mu}\ \phi_{mu} \tag{16}$$

where $e_{d,DBE}$ = Compressive strain demand on the masonry in the beam at the Design Basis Earthquake.
e_{mu} = Expected Maximum usable compressive strain in the masonry.
ϕ_{mu} = Capacity reduction factor for maximum usable strain.

The design equation for the ultimate strain and, for example, a user/client defined MCE ground motion is

$$e_{d,MCE} \leq e_{uu}\, \phi_u$$

$e_{d,MCE}$ = Compressive strain demand on the masonry in the shear wall at the Maximum Credible Earthquake.

e_{uu} = Expected ultimate compressive strain in the masonry.

ϕ_u = Capacity reduction factor for ultimate compressive strain in the masonry.

Table 2 provides an illustration of how the strain related limit states are linked to the ground motion to formulated a performanced based design criteria.

TABLE 2 DESIGN PERFORMANCE LIMITS

Limit State	Design Earthquake	Limit State
1	Maximum Probable (50% in 50 years)	Tensile strain in steel equal to yield strain in steel or alternately the moment equal to the yield moment.
2	Design Basis (10% in 50 years)	Compressive strain in masonry equal to maximum usable strain
3	Maximum Capable (10% in 100 years)	Compressive strain in masonry equal to ultimate compressive strain.

Masonry design in high seismic regions must be viewed from a strain perspective and not a force perspective. This is a fundamental departure from the existing viewpoint of most structural engineers who view design from a force perspective. It is only by quantifying the response in terms of limit states defined in terms of states of strain can be how masonry structures unwind with layer earthquake ground motion.

SUMMARY

The design of masonry structures in high seismic regions requires that we focus on rational methods to quantify performance. This requires that sophisticated and simple analysis be calibrated with experiment and that a probabilistic framework be selected to quantify expected values and uncertainties.