Assessment of Seismic Behaviour of Buildings in Regions Of Moderate Seismicity

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ABSTRACT

This paper describes a methodology that can be used to determine the level of seismic risk to buildings in a modern city with densely populated high-rise buildings in a region of moderate seismicity. It further explores cost effective methods for seismic risk mitigation. Firstly, the level of seismic hazard and site response effects need to be quantified. Typical building types are then identified and their response to seismic ground motion is quantified by various applicable methods of analysis ranging from linear response spectrum analysis to non-linear static or dynamic time-history analysis. The HAZUS® methodology is used as a basis for quantifying seismic risk. However, it is realized that the behaviour of the high proportion of high-rise reinforced concrete buildings up to about 60 stories cannot be adequately characterized using the default HAZUS® building capacity and fragility curves. Nonlinear seismic response analysis of high-rise buildings is carried out to extend the default HAZUS® building capacity and fragility curves to be applicable to these buildings. An example is given to illustrate a solution to overcome the difficulty encountered when determining the damage medians for the more severe damage states based on nonlinear seismic analysis results. A Geographical Information System grid analysis is then proposed to quantify the seismic risk to the whole building stock in the city in terms of degree of building damage, the direct economical loss, and casualties. Finally, cost benefit analyses can be used to determine whether the introduction of a moderate level of seismic design with some detailing rules for ductility would be cost effective.

INTRODUCTION

In many modern cities in regions of moderate seismicity many people live and work in high-rise buildings. Often the relevant codes of practice for building design do not require any seismic considerations. The combination of the extremely densely populated urban environment, the economic significance of the region and the potential vulnerability of the building stock to earthquakes warrant further investigation of the seismic risk to these buildings.

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This paper describes a methodology that can be used to determine the level of seismic risk to buildings in such a city as a result of potential future seismic ground motion. It further describes a cost-benefit analysis method to determine whether the introduction of a moderate level of seismic design with some detailing rules for ductility would be cost effective.

SEISMIC HAZARD AND SITE RESPONSE EFFECTS

The seismic hazard needs to be quantified in terms of uniform hazard response spectra having a 50%, 10% and 2% chance of being exceeded in 50 years for bedrock sites using the standard procedure suggested by Cornell (1968). The basis of the input and the typical type of results are described in Free et al. (2004). Site response effects can then be allowed for by reviewing existing borehole logs and classifying them using the NEHRP site classification system (FEMA 1997). In this system the stiffness of the upper 30m of soil or rock is considered and classified into Site Class A for hard rock to Site Class E for soft soil. The paper by Pappin et al. (2004) describes the application of such a classification procedure in some detail and Figure 1 shows an extract of the resulting Site Class zoning map that was produced. It can be seen that only Site Classes B, C, D and E were applicable to that particular location.

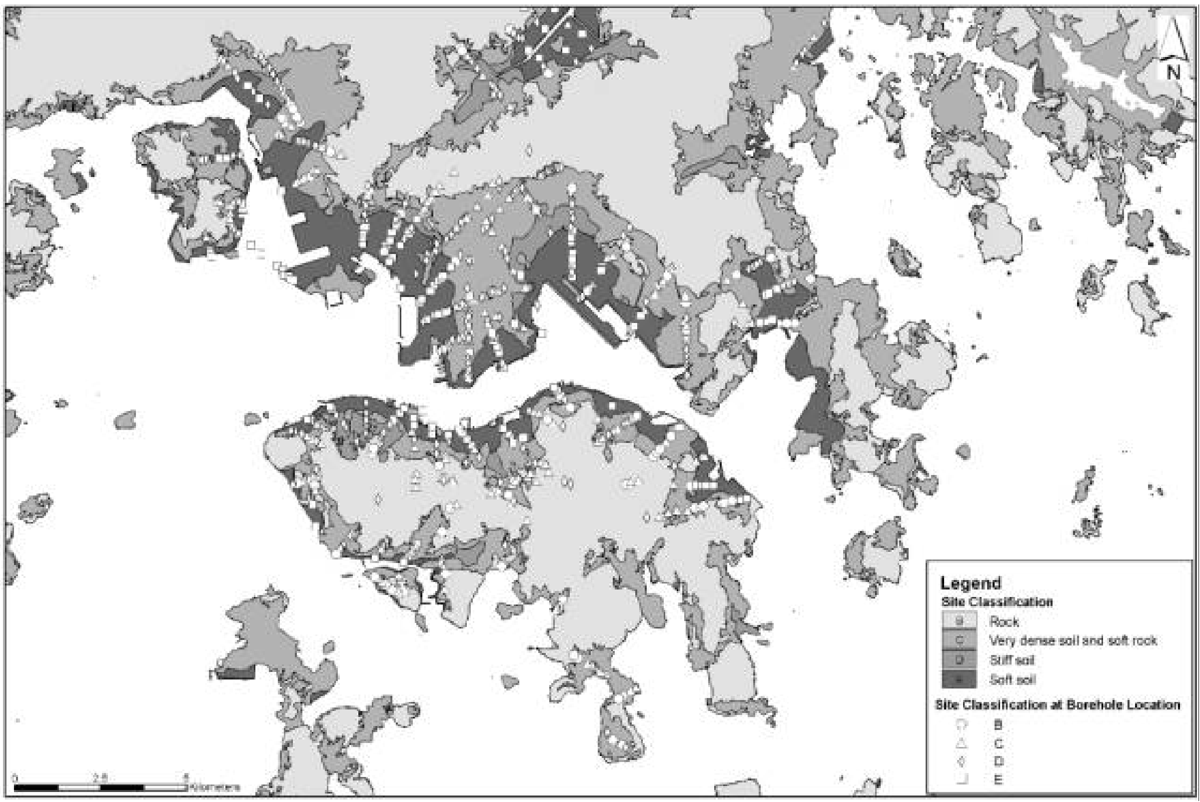


Fig. 1. Extract of a site class zoning map (Pappin et al., 2004)

Having classified the boreholes a representative selection of each Soil Class should be subjected to one dimensional site response calculations and response spectral amplification factors determined as a function of period for a range of bedrock outcrop ground motion corresponding to the 50%, 10% and 2% probabilities of being exceeded in 50 years. For all ground motion levels both the median amplification and standard deviations of the amplification factors need to be determined as a function of period. Pappin et al. (2004) report that for a certain period each of these factors could be expressed as a function of the bedrock response spectral value at that period. While the median spectral amplification factors were dependent on the Site Class it was found that the standard deviations could be expressed as a function of the median spectral amplification factor and were independent of Site Class.

Figure 2 shows an example of calculated horizontal uniform hazard response for ground motion levels having a 50% and 2% probability of being exceeded in 50 years. The bedrock outcrop response spectra are shown as solid black lines and the ground surface response spectra for Site Classes B to E are shown as lines with symbols. For each Site Class the median values are shown as solid lines and the median plus two standard deviations are shown as dashed lines. For reference the bedrock spectra have peak horizontal ground accelerations of about 5%g and 35%g for the 50% and 2% in 50 year probability values respectively. The 5% damped peak response spectral acceleration values occur at a period of about 0.1 seconds and are about double these values (see Free et al., 2004).

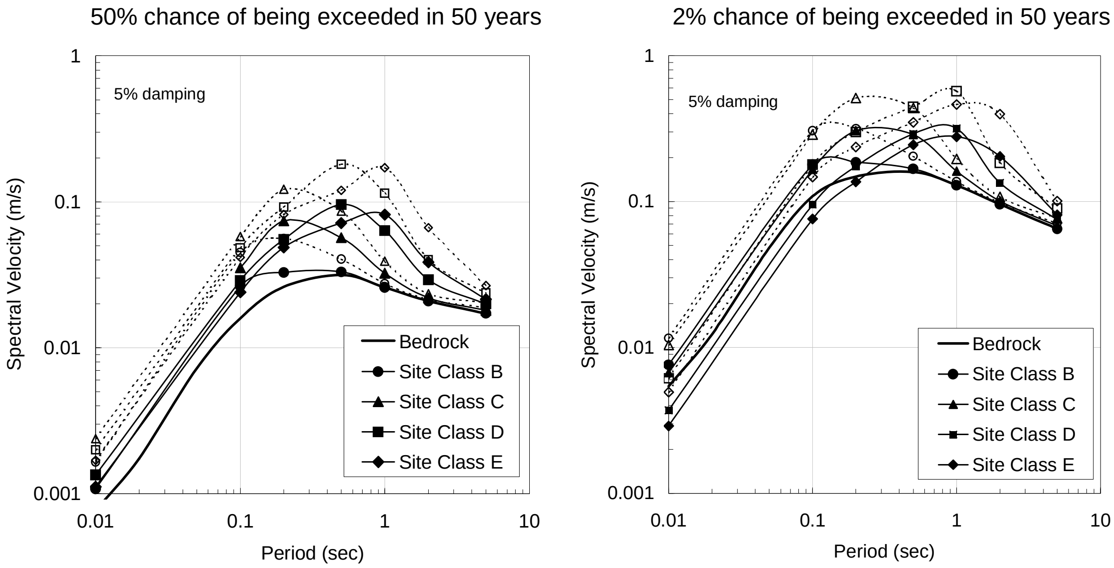


Fig. 2. Horizontal velocity uniform hazard response spectra for various Site Classes (Pappin et al., 2004)

RESPONSE OF BUILDINGS TO SEISMIC GROUND MOTION

The methodology suggested to determine the response of buildings to seismic ground motion is based on the HAZUS recommendations, FEMA (2003). This methodology includes default values to characterize the response of a range of building structural types up to a height of about stories. Where these HAZUS building categories are applicable, modifications to the HAZUS default values may still be necessary to account for the wind loads that buildings are designed to resist. In addition the building stock in many modern cities comprises many buildings with 15 to 60 stories and additional building categories need to be introduced to cater for these.

Inventory of the Building Stock

As a first step an inventory of buildings needs to be compiled, ideally in a Geographical Information System (GIS) database. The inventory needs to record the building footprint, total floor area, the number of stories, the building usage and the number of occupants in day and night time. It is inevitable that the data will not be complete and some rules will be need to be derived, based on building usage, to derive structural type and occupancy rate. The building monetary value also needs to be estimated in terms reconstruction cost.

Generally the modern building stock is dominated by reinforced concrete buildings with some unreinforced masonry buildings in the older low rise buildings. It is suggested that the building heights be divided into 5 ranges as follows:

- 1 to 2 stories and 3 stories, generally referred to as Low-rise (L) buildings,
- 4 to 7 stories, referred to as Mid-rise (M) buildings,
- 8 to 15 stories, referred to Intermediate high-rise (I) buildings,
- 16 to 30 stories, referred to as High-rise (H) buildings and
- 31 to 60 stories, referred to as Very high-rise (V) buildings.

A suggested breakdown of building structural systems is as follows:

Unreinforced Masonry Bearing Walls (URM): These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. For older buildings built before say 1900, the majority of floor construction may well comprise wood sheathing supported by wood framing whereas in newer buildings, the floors may be cast-in-place concrete supported by the unreinforced masonry walls and/or concrete interior framing. The perimeter walls, and possibly some interior walls, are often unreinforced masonry and may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels.

Reinforced Concrete Moment Resisting Frames **(CF):** These buildings have a reinforced concrete frame and develop their stiffness by full or partial moment connection of these frames. Many buildings are likely to have a reinforced concrete core that transfers a significant part of the lateral loading to the ground.

Concrete Frame Buildings with Infill Walls (CFIW): These buildings have a reinforced concrete frame and develop their stiffness by full or partial moment connection of these frames, as for CF buildings, but with infill walls. In these buildings, the shear strength of the columns, after cracking of the infill, may limit the semi-ductile behavior of the system. In older buildings the infill will typically be unreinforced masonry. In newer buildings concrete infill with a single layer of steel mesh reinforcement is typically used. These walls are not designed to take any of the lateral load however.

Concrete Shear Walls (CSW): The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low but reinforcing is light. In newer buildings, the shear walls are often limited in extent, leading to concerns about boundary members and overturning forces.

The HAZUS Earthquake Loss Estimate Methodology for Buildings

It is proposed that the HAZUS methodology (FEMA, 2003) is used as the basis for determining the seismic response of the building stock. The methodology is summarized in Figure 3. Default values for building response are only directly applicable to buildings up to about 15 stories high. Calibrating the HAZUS method for the building categories in a particular location and extending the method to cover buildings above 15 stories is discussed later. In HAZUS, the general building stock represents typical buildings of a building type designed to either High-Code, Moderate-Code, or Low-Code seismic standards, or not seismically designed (referred to as Pre-Code buildings). For the application of the method to buildings to many cities in regions of moderate seismicity, only the Low-Code and Pre-Code building types are likely to be appropriate.

Building damage states

In HAZUS the building damage is predicted in terms of one of four ranges of damage or "damage states": Slight, Moderate, Extensive, and Complete. For example, the Slight Damage state extends from the threshold of Slight Damage up to the threshold of Moderate Damage. General descriptions of these damage states are provided for the model building types with reference to observable damage incurred by structural building components. Damage predictions resulting from this physical damage estimation method are expressed in terms of the probability of a building being in any of these four damage states. As an example the description of Slight, Moderate, Extensive, and Complete Structural Damage is provided here for reinforced concrete frame buildings (CF).

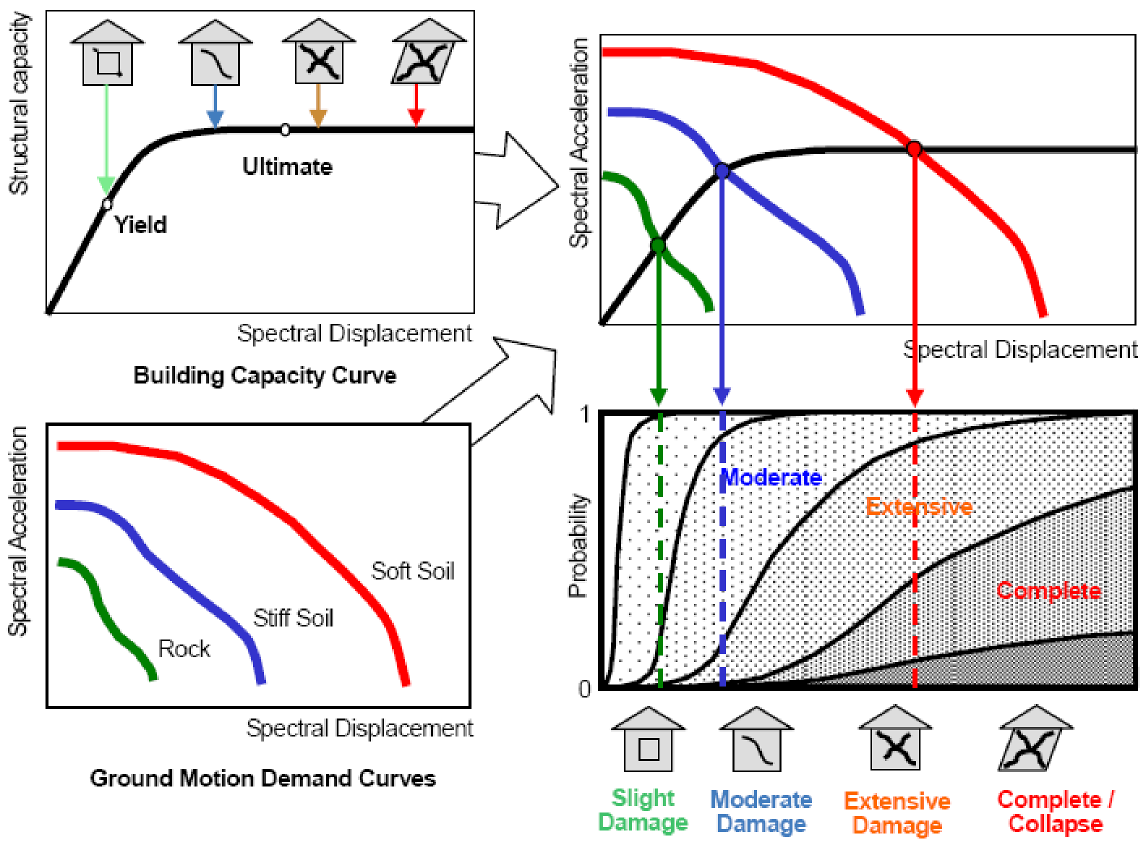


Fig. 3. HAZUS earthquake loss estimate methodology for buildings

Slight Damage: Flexural or shear type hairline cracks in some beams and columns within or near joints.
Moderate Damage: Most beams and columns exhibit hairline cracks. In ductile frames some of the frame elements have reached yield capacity indicated by larger flexural cracks (>4mm) and some concrete spalling. Non-ductile frames may exhibit larger shear cracks and spalling.
Extensive Damage: Some of the frame elements have reached their ultimate capacity, indicated in ductile frames by larger flexural cracks, spalled concrete and buckled main reinforcement. Non-ductile frame elements may have suffered shear failures or bond failures at reinforcement splices, or broken ties or buckled main reinforcement in columns which may result in partial collapse.
Complete Damage: Structure has collapsed or is in imminent danger of collapse due to brittle failure of non-ductile frame elements or loss of frame stability. About 13% (low-rise), 10% (mid-rise) or 5% (intermediate high-rise) of the buildings with Complete Damage are expected to have collapsed.

Building capacity curves

Except for a few brittle systems and acceleration-sensitive elements, building damage is primarily a function of building distortion or drift, rather than force. In the inelastic range of building response, increasingly larger damage would result from increased building drift although lateral force would remain constant or decrease. Hence, successful prediction of earthquake damage to buildings requires reasonably accurate estimation of building drift response in the inelastic range. While linear elastic analysis methods can be used to estimate drift they will under-predict the building drift as the buildings respond in-elastically to earthquake ground shaking of interest for damage prediction.

Building capacity curves, used with the demand spectrum as recommended by the HAZUS methodology, provide a simple and reasonably accurate means of predicting inelastic building displacement response for damage estimation purposes.

A building capacity curve, also known as a pushover curve, is a plot of a building's lateral load resistance as a function of a characteristic lateral displacement or drift, i.e. a force-deflection plot. It is derived from a plot of static-equivalent base shear versus a measure of the building displacement or drift, for example at roof level. In order to facilitate direct comparison with earthquake demand (i.e. overlaying the capacity curve with a demand spectrum), the force (base shear) axis is converted to spectral acceleration and the drift axis is converted to spectral displacement. Such a plot provides an estimate of the building's "true" deflection (drift response) for any given earthquake demand spectrum.

The parameters used by HAZUS to define the capacity curve are shown in Figure 4. Three control points, that define the building capacity curve, are as follows:

- *Design Point* (D_D,A_D): representing the expected nominal strength for the building. In the HAZUS default parameters wind design is not considered in the estimation of design capacity.
- *Yield Point* (D_Y,A_Y): representing the true lateral strength of the building considering redundancies in design, conservatism in code design calculation methods and true (not nominal) strength of materials.
- *Ultimate Point* (D_U,A_U): representing the maximum strength of the building when the global structural system has reached a fully plastic state. Ultimate capacity implicitly accounts for loss of strength due to shear failure of brittle elements. Typically, buildings are assumed to be capable of deforming beyond their ultimate point without loss of stability, but their structural system provides no additional resistance to lateral earthquake force.

Up to the Yield Point, the building capacity curve is assumed to be linear with stiffness based on an estimate of the true period (T) of the building. As this point represents the condition where the building is beginning to yield and experience permanent damage the true period is typically longer than the measured small deformation period of the building. For concrete frame buildings HAZUS recommends that the true period be assumed as 1.33 times larger than the code specified period. For concrete shear wall buildings and concrete frame buildings with infill walls this ratio is about 1.5 and for unreinforced masonry the ratio increases to about 2. From the Yield Point to the Ultimate Point, the capacity curve changes in slope from an essentially elastic state to a fully plastic state. The capacity curve is assumed to remain plastic past the Ultimate Point. As shown in Figure 4 the control points are defined by the following parameters:

C_S design strength coefficient (fraction of building's weight),
α_1 fraction of building weight effective in pushover mode,
γ "over-strength" factor relating "true" yield strength to design strength,
λ "over-strength" factor relating ultimate strength to yield strength, and
μ "ductility" factor relating ultimate displacement to λ times the yield displacement over-strength.
T Building period (seconds)

In HAZUS the design strength coefficient, C_S, is approximately based, on the lateral-force design requirements of current seismic codes. Its value is a function of the seismic zone location, the site soil condition, type of lateral-force-resisting system and building period. Figure 5 shows the HAZUS capacity curves for buildings with no seismic design provisions for the range of buildings discussed previously. The three diagonal dashed lines show how the building period varies as a function of the spectral acceleration and displacement. As expected the building period is constant before the yield point is reached.

Building structure fragility curves

The probability of being in, or exceeding a given damage state is modeled as a cumulative lognormal distribution. The lower part of Figure 6 shows an example set of fragility curves for a Pre-Code low-rise Concrete Frame building determined using the default HAZUS values. Median values of structural component fragility are based on building drift ratios that describe the threshold of damage states. Damage-state drift ratios are converted to spectral displacement using;

$$\overline{S}_{d,Sds} = \delta_{R,Sds} \cdot \alpha_2 \cdot h \tag{1}$$

where: $\delta_{R,Sds}$ is the drift ratio at the threshold of structural damage state, ds,

α_2 is the fraction of the building (roof) height at the location of pushover mode displacement, = 0.75 for low-rise to 0.6 for high-rise buildings.

h is the typical roof height, of the building type.

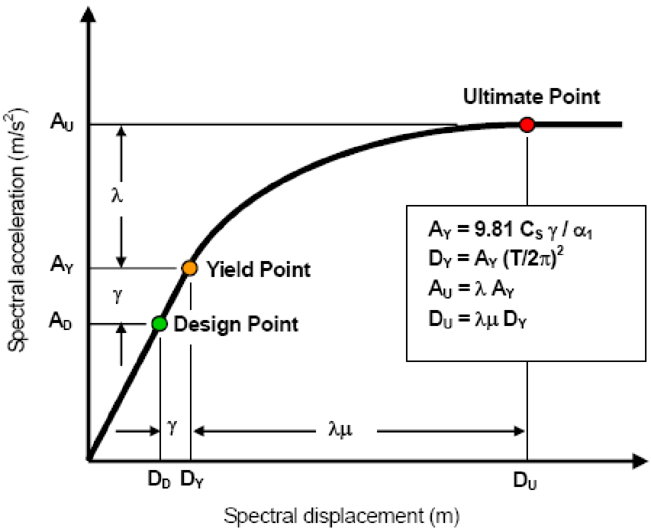


Fig. 4. HAZUS capacity curve definition

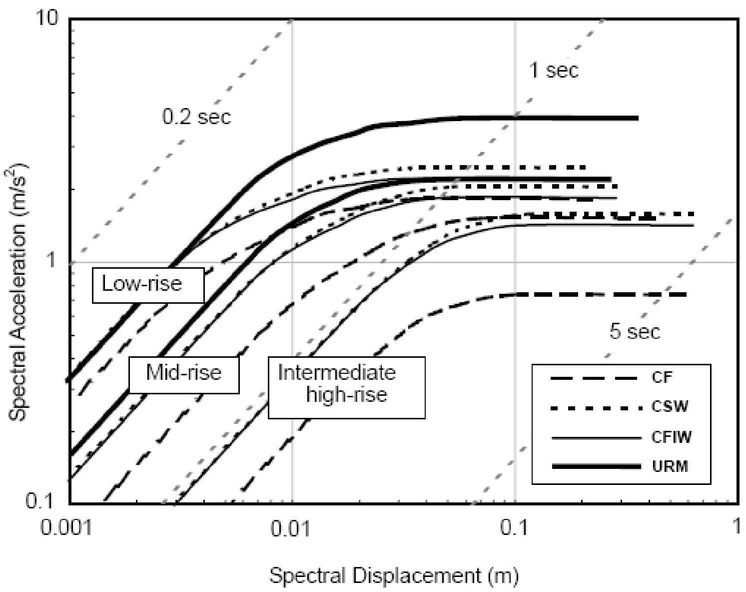


Fig. 5. HAZUS default building capacity curves for pre-code buildings

A lognormal standard deviation is used to describe the total variability for structural damage state. In HAZUS it is often taken to include the variability of the ground motion as well as that of the building stock and also allows for variability of the capacity curve. A value of 0.5 for the natural log is used in Figure 6 and, as discussed later, is a reasonable value for use with uniform hazard spectra that already contain specific allowance for the known variability of the input ground motion.

To illustrate how the capacity curve, the fragility curve and the demand curve interact the upper part of Figure 6 shows the default HAZUS capacity curve and a set of three demand spectra. These are for illustration only and approximately correspond to the demand spectra that envelope those shown in Figure 2. For each intersection point a line is drawn down to the fragility curves in the lower diagram. The damage predicted to arise from the 10% in 50 year ground motion, for example, is about 30% of the building stock will experience Slight Damage, 50% Moderate Damage and 5% Extensive Damage.

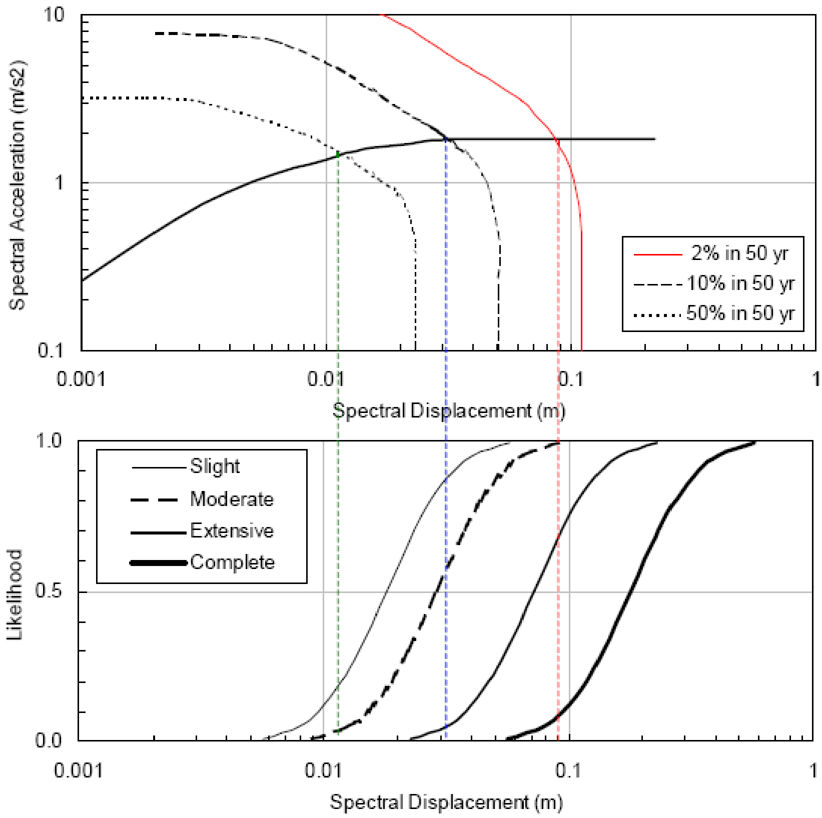


Fig. 6. Example for low-rise concrete frame (CF) buildings using the default HAZUS values for pre-code buildings

It should be noted that HAZUS gives a method to reduce the demand spectra for additional damping that the building will experience as it reaches the ultimate capacity point. For the low ductility buildings that often exist in regions of moderate seismicity however this effect is not likely to be that significant however and a structural damping of 5% will often be adequate.

Building structure seismic analysis methods

Various analysis methods can be used as follows:

Linear dynamic analysis

For this analysis method the building design seismic forces, the distribution of these forces over the height of the building, and the corresponding internal forces and displacements are determined using a linear-elastic, dynamic analysis. These time-history analyses involve a step-by-step analysis of the mathematical model of a building using digital earthquake time-histories. The forcing function for the analysis can be an acceleration time-history that has a response spectrum similar to the 2% in 50 year ground motion as shown for example in Figure 2. For a building that does not yield the linear elastic time-history analyses accurately predict element stresses and displacements. If the building response is anticipated to yield and be beyond the elastic region, the predicted displacements will be too small and the predicted internal forces will exceed those that would be obtained in the yielding building. For small levels of non-linearity the predicted displacements will still be quite close to the correct result but the internal forces may be too large.

Non-linear dynamic time-history analysis

Modeling the full non-linear behavior of certain building components is required to understand failure mechanisms. The program *Oasys* LS-DYNA or equivalent can be used to perform non-linear time-history analyses. The static dead and live loads are applied to a mathematical model of the building and the stresses within the structural element are determined. An earthquake time-history is then applied to the base of the model and a dynamic analysis carried out. LS-DYNA can model geometric non-linearity, material non-linearity and inelastic behavior and higher mode effects. In addition, it is capable of capturing local instability in the structure and assessing the "Yielding Surface" defining the relationship between the two axes of bending and the axial force. This is important for corner columns common to two intersecting frames.

Non-linear static pushover analysis

A pushover analysis is used to determine the capacity of a structure to resist lateral seismic forces. A mathematical model of the building is first subjected to the static gravity loads to determine the stresses within the structural elements and then the model is pushed horizontally with increasing static lateral load until it is unable to support any additional loading. The amount of force the building is able to sustain is plotted versus the relative displacement at the roof level. The resulting pushover curve is a plot of the base shear versus drift and is equivalent to the capacity curve used by the HAZUS methodology. The pushover analysis procedure is defined and detailed in ATC 40, FEMA-273, FEMA-274 and FEMA-356. To be consistent ACI 318 should be used to estimate member strengths and the HAZUS guidelines (FEMA, 2000), used to derive the capacity curves. The pushover curves should be calculated for each direction of response and the flexibility of all elements and components, including infill walls that contribute significantly to the building response incorporated into the analyses. The pushover models are usually three-dimensional in order to capture any torsional behavior of the response.

Linear base shear force analysis

This is the simplest form of analysis and forms the basis of most seismic codes of practice. It comprises using the fundamental period of the building with an acceleration response spectrum to give an estimate the maximum base shear as a ratio of the building weight that the building will experience under the action of the design seismic event. The fundamental periods of the buildings can be estimated using the conventional code formula adjusted as recommended by HAZUS. If a time-history analysis was carried out the building period from that analyses can be used.

An interesting product of the base shear analysis, when compared with the base shear resulting from the linear time-history analyses, is a measure of the significance of higher mode responses of the building. In the HAZUS methodology the modal parameter α_1 describes the amount of the building mass that is likely to participate in the fundamental mode pushover analysis. If the building response is dominated by this mode, with only small input from higher modes, it would be expected that the base shear resulting from the time-history dynamic analyses would be similar to that predicted by the simplified base shear analysis method multiplied by the α_1 value given by HAZUS.

For buildings with longer periods however higher mode effects mat become significant and it can be observed that the base shear predicted by the linear dynamic time-history analysis is significantly greater than that implied by the response of the structure to the first mode only. This can especially the case for ground motions whose response spectra have low values at higher periods such as those indicated in Figure 2. In the building damage assessments of high-rise buildings this effect from higher mode effects must be taken into account. This adjustment can be achieved by using a base shear factor

η that relates the calculated peak base shear to that predicted by the first mode response only. The α_1 factor is replaced by the base shear factor η. For low-rise and mid-rise buildings it is set equal to the appropriate HAZUS α_1 factor, but for higher buildings it may become significantly larger than unity.

Design wind load

The wind forces that the buildings have been designed for can be used to derive the minimum lateral yield capacities of the buildings. To derive the expected building design capacity it is necessary to allow for any material factors that are inherent in the structural codes and also to allow for the expected difference between the expected material strength and the characteristic strength likely to be used in the original calculations. The resulting yield capacity is used to derive a yield capacity coefficient by dividing this expected yield capacity by the seismic weight of the building. The coefficient, referred to here as C_{SY}, is the same as $C_S * \gamma$ in the HAZUS methodology.

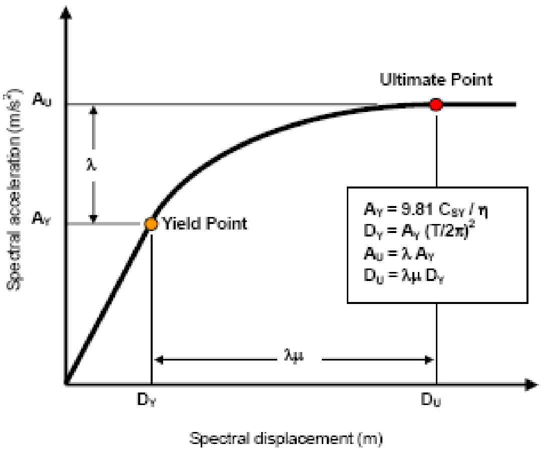


Fig. 7. Definition of capacity curve

Definition of building capacity curves

Figure 7 shows how the building capacity curves are defined using the C_{SY} and the η coefficients suggested above and the over-strength, λ, and ductility, μ, coefficients already defined by the HAZUS methodology. Comparison with Figure 4 shows that the definition is very similar to that used by the HAZUS methodology and for suitable building classes, could be directly defined using the HAZUS coefficients. For low-rise to intermediate high-rise buildings it is necessary to calibrate and possibly modify the default HAZUS coefficients to ensure they are appropriate to the buildings being characterized. For high-rise and very high-rise buildings not covered by the default HAZUS values additional coefficients need to be generated.

For building categories that correspond to the HAZUS default classifications, comprising low-rise to intermediate high-rise buildings up to about 15 stories, the following procedure is recommended:

- For a selection of buildings the capacity curves can be derived from a non-linear pushover analysis.
- For most buildings an estimate of the lateral shear capacity can be derived from the wind loads as a check on the HAZUS values. The design wind loads are assumed to give a lower bound indication of the capacity of the building. Wind loads depend on the height, plan area and shape of the building and can be substantially different in each direction. For the buildings the wind load is determined and used to assess the minimum expected lateral yield capacity as explained previously. By combining the expected yield capacity with an estimate of the building period a capacity curve can be generated using the default HAZUS over strength and ductility coefficients.

For high-rise buildings that do not correspond to a HAZUS classification the following procedure is recommended:

- Carry out linear elastic dynamic time-history analyses.
- If the time-history calculation shows that the main building structure does not experience yield no further analysis is required. If significant yield is predicted however, non-linear time-history analyses or pushover analyses are required. It must be emphasised that the pushover method, in a similar way to the HAZUS procedure, assumes the first mode response is generally sufficient to represent the behaviour of the building. Therefore non-linear time-history dynamic analyses are necessary to confidently predict the response of high-rise buildings to extreme ground motions.
- For high-rise buildings wind loads can be very significant and can be used to derive the expected yield capacity of the building. As for lower rise buildings this needs to be combined with an estimate of the building period to derive the building capacity curve. Other coefficients need to be assessed by suitable extrapolation from the lower-rise buildings.

Definition of building fragility curves

As illustrated in Figure 6 Fragility Curves are functions that describe the probability of the buildings within each category reaching, or exceeding, the various damage states for a given building response. This family of curves has inherent properties that can be assessed from earthquake damage data. In a seismic risk study carried out previously for the UK Government (Arup, 1993), the prime building type was unreinforced masonry. Damage data had been obtained from a wide variety of damage surveys as reported in Coburn and Spence (2002) and these are shown in Figure 8. Each damage survey location is represented by a series of points vertically above each other at a single location on the horizontal Intensity scale. It can be seen that the various levels of damage are interrelated. For example if 50% of the buildings have experienced Heavy Damage or more (at an Intensity ψ value of 10) it is observed that about 25% have experienced at least Partial Destruction and about 80% at least Moderate Damage. All buildings have experienced at least Slight Damage. Given the interrelation of curves vertically however means that the width (or lateral spread) of the curves and their relative horizontal position are mutually dependent. The family of curves needs therefore to be viewed as a whole and the horizontal axis chosen to best fit the observed levels of damage.

The shape of the fragility curves should reflect the variability of buildings within the same category. For a particular category of buildings where all buildings are very similar it would be expected that the variability should be very low. In general damage surveys are a reasonable guide for deriving the inherent variability of buildings and the variance derived using these methods should be adhered to unless there is a clear reason to deviate. The variability of the natural logarithm of 0.47 to 0.5 specified in HAZUS for Low-Code and Pre-Code buildings is a reasonable starting point. A value of 0.5 accords well with that shown for unreinforced masonry as shown in Figure 8 and is recommended. As noted previously Figure 6 is based on a value of 0.5.

As described above HAZUS presents median values of the fragility curves defining each damage state. For buildings where pushover analysis is carried out a check on the HAZUS values can be made. This can be done following the HAZUS methodology, FEMA (2000), which defines two sets of criteria for each damage state. The criteria are shown in Table 1 and refer to the percentage of elements that have reached various points on their non-linear response curve. Figure 9 shows the schematic response curve where Point B is yield and point C is the ultimate capacity. Criteria Set 1 in Table 1 refers to the fraction of components that have reached the ultimate capacity (Point C) and Criteria Set 2 the fraction of components that have reached yield.

The fraction of components is weighted to the replacement value of the components and if any one group of components can lead to the building collapse (e.g. the columns on the ground floor) they will represent the full replacement value. The Criteria Set that gives rise to the lowest drift is the controlling case.

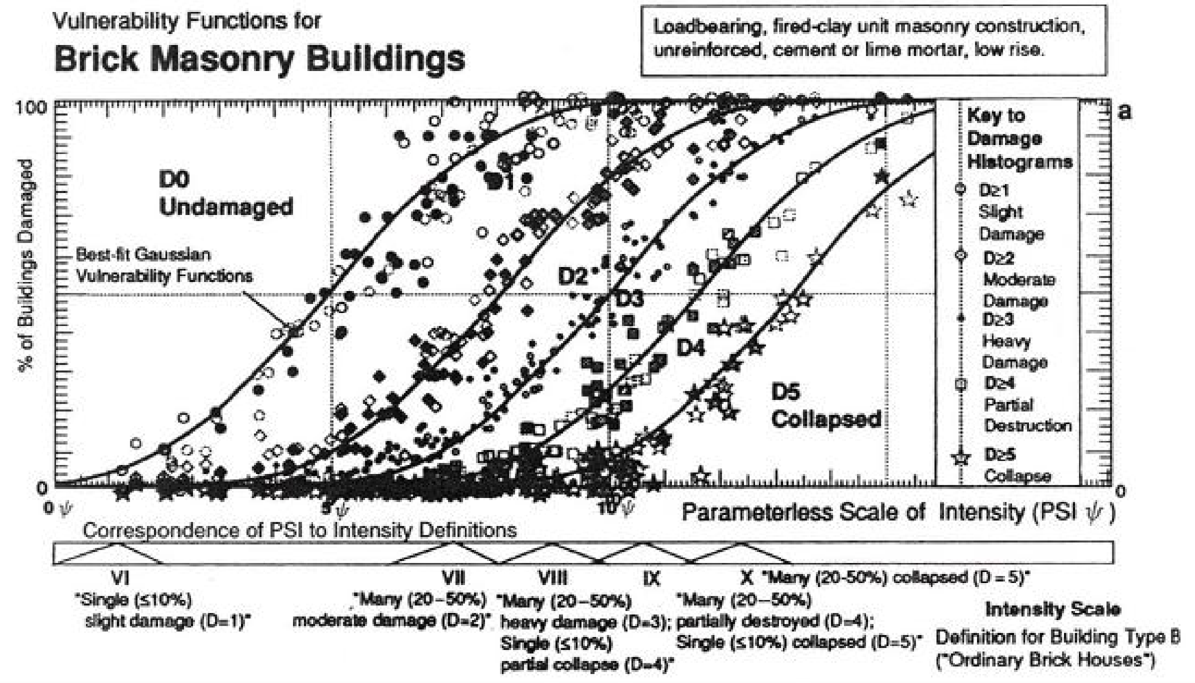


Fig. 8. Observed fragility curves for ordinary brick buildings (Coburn and Spence, 2002)

Table 1 General Guidance for Relating Component (or Element) Deformation to the Average Inter-story Drift Ratios of Structural Damage State Medians (FEMA, 2000)

Damage State	Component (Criteria Set No. 1)[1]			Component (Criteria Set No. 2)[1]		
	Fraction[2]	Limit[3]	Factor[4]	Fraction[2]	Limit[3]	Factor[4]
Slight	> 0%	C	1.0	50%	B	1.0
Moderate	≥ 5%	C	1.0	50%	B	1.5
Extensive	≥ 25%	C	1.0	50%	B	4.5
Complete	≥ 50%	E	1.0 - 1.5[5]	50%	B	12

1. The average inter-storey drift ratio of structural damage state is lesser of the two drift ratios defined by Criteria Sets No.1 and No.2 respectively.
2. Fraction defined as the replacement value of components at the limit divided by the total replacement value of the structural system.
3. Limit defined by the control points of Figure 9 and the acceptance criteria of NEHRP Guidelines.
4. Factor applied to average inter-storey drift of structure at deformation (or deformation ratio) limit to calculate average inter-storey draft ratio of structural damage state median.
5. Complete factor is largest value in the range for which the structural system is stable.

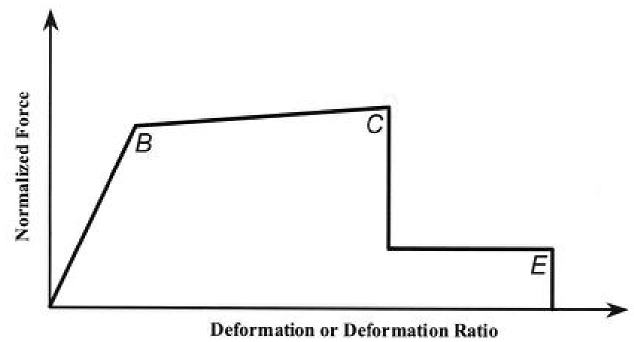


Fig. 9. Idealized force deformation curve, FEMA (2000)

Figure 10 shows a schematic diagram for relating the pushover curves to the Damage State median values. The Slight Damage point is seen to be near to the first significant yield point and the Extensive Damage point slightly beyond the highest capacity point calculated by the pushover analysis. For buildings that do not correspond to a HAZUS classification (i.e. high-rise buildings) linear time-history analyses can be used to determine the median value for the threshold of Slight Damage by assuming this occurs at a displacement a little greater than that required to cause the first yield of any part of the primary structure. For Moderate, Extensive and Complete Damage the threshold displacements can be assumed to occur at multiples of the yield value respectively. HAZUS generally recommends ratios of 1.5, 4.5 and 12 as implied in Table 1.

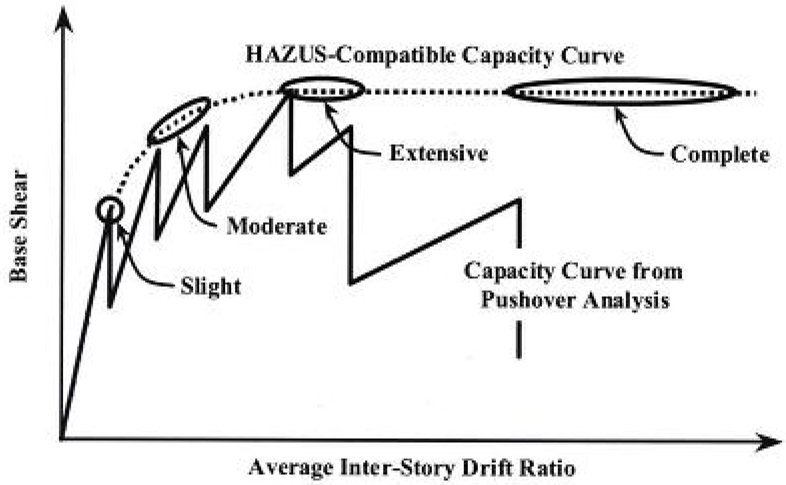


Fig. 10. Example damage state medians from “Saw tooth” Pushover curve, FEMA (2000)

Allowances can also been made for the effects of pounding where existing buildings are built touching each other and for high rise shear wall buildings that are built on transfer plates that are only supported by a column frame system.

Deriving building earthquake damage fragility curves from nonlinear analysis

The need

The HAZUS® recommended default building capacity curves and fragility curves are applicable to low-rise, medium-rise and intermediate high-rise (termed high-rise in HAZUS®) buildings only. For high-rise and very high-rise buildings, the capacity curves and fragility curves must be established by extending the HAZUS® recommended default curves, or alternatively by carrying out nonlinear static pushover analysis.

The analysis method

The methodology and procedure for carrying out nonlinear static pushover analysis are well-documented in ATC 40 and FEMA 356. For the purpose of earthquake loss estimate, the best estimate structural component stiffness and strength capacity values are adopted to build nonlinear seismic analysis models. Such a model has the median stiffness and strength properties of the type of buildings represented by the building selected for analysis. If under a specified level of ground motion, a building’s seismic response is linear or nearly linear, then its behaviour depends primarily on its stiffness and strength properties. In this case, the behaviour of the idealised nonlinear analysis model can adequately simulate the median response of the type of buildings it represents. Hence, it is relatively easy to establish the median intertorey drift ratios (spectral displacements) for the yielding, the slight and the moderate damage states.

The problem

However, difficulties will be encountered if the structure is shaken (pushed) well into the inelastic range. In this case, the behaviour of the building depends not only on the stiffness and strength properties, but also critically on the structural components' inelastic deformation (ductility) capacities, namely their ability to develop plastic deformation while maintaining their lateral-load resisting capacity. Presently, the structural components' inelastic deformation (ductility) capacity for use in nonlinear analysis is commonly obtained from data recommended in ATC 40 and FEMA 356. Because ATC 40 and FEMA 356 are design guideline documents, the inelastic deformation capacity (acceptance criteria) recommended in these two documents are inherently conservative. A problem and difficulties arise from this point for determining the damage medians for the more severe damage states (extensive and complete), because the ATC 40 and FEMA 356 recommended nonlinear modelling parameters and inelastic deformation acceptance criteria are not the median values.

Hence, beyond the moderate damage state, the pushover analysis results cannot be used directly for determining the damage medians for the extensive and the complete damage states. The latter segments of a building's capacity curve obtained from pushover analysis underestimate the structure's inelastic deformation capacity. While this conservatism is desirable for the purpose of design of new buildings as well as assessment and rehabilitation of existing buildings, it is nevertheless unsuitable for loss estimate. Damage functions should predict losses without bias. Hence, from the point of view of building seismic loss estimate, conservatism in seismic design codes and guideline documents must be removed.

The above argument can be better understood by examining the complete damage state (also termed near collapse, or collapse prevention). The ATC 40 and the FEMA 356 recommended inelastic deformation acceptance limits for collapse prevention certainly are not the median values. It is not the intension of ATC 40 and FEMA 356 to provide median acceptance limits whereby 50% of the buildings just meeting the collapse prevention acceptance criteria would collapse and the remaining 50% do not collapse.

A solution

A solution proposed in this study to the above described problem is to use the capacity spectrum from the pushover analysis, but recognising that spectral displacement corresponding to the near collapse point is related to a much lower probability than the median value of 50%. HAZUS® suggests that most engineers would likely consider the ATC 40/FEMA 356 collapse prevention criteria to correspond to a probability of between 1% to 10%. Therefore, this study has proposed the following steps for establishing the damage medians for the extensive and the complete damage states:

- Use the spectral displacement value corresponding to the near collapse point on the building's capacity curve from the pushover analysis
- Relate this spectral displacement to a much lower probability value on the fragility curve, between 1% to 10%, rather than the median value (50%), to establish the damage median for the complete damage state
- The median spectral displacement for the extensive damage state can be determined by the location of the HAZUS® ultimate capacity point $D_u = 2D_y \dfrac{A_u}{A_y}$ or the proportions between various damage medians suggested in HAZUS® rule set No. 2 as shown in Table 1.

An example

A real example is given herein to illustrate the solution method proposed in this study. Figure 11 shows the deformed shape of a high-rise reinforce concrete moment frame – shear wall (core) building during the pushover analysis. Figure 12 presents the capacity spectrum obtained from the nonlinear pushover analysis, with the yielding point, the median points for the slight and the moderate damage states and the near collapse point identified on this curve. Figure 13 illustrates the process for determining the fragility curve for the complete damage state using the near collapse point in the capacity curve from the pushover analysis.

CALCULATION OF SEISMIC RISK

To determine the risk to buildings from seismic ground motion a series of calculations must be carried out to determine the level of damage to all the buildings in the city. In this risk calculation the uniform hazard bedrock spectra having probabilities exceedance of 2%, 10% and 50% in a 50 year period, are imposed on the building stock. The resulting damage is evaluated in terms of square meters of floor area to experience Slight, Moderate, Extensive and Complete Damage. The resulting overall cost and number of casualties are also calculated. By integrating the results over the different probabilities of being exceeded the overall annual cost to buildings as a whole can be estimated. It must be emphasized that these calculations are not to predict the effects of a single earthquake but rather the aggregated effect of all the future possible earthquake events.

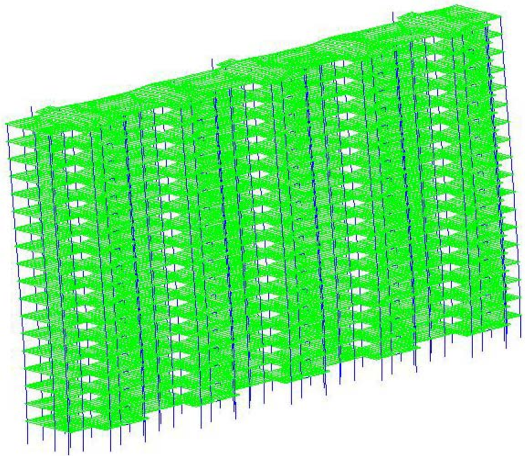


Fig. 11. Deformed shape of a high-rise reinforced concrete moment frame – shear wall (core) building during pushover

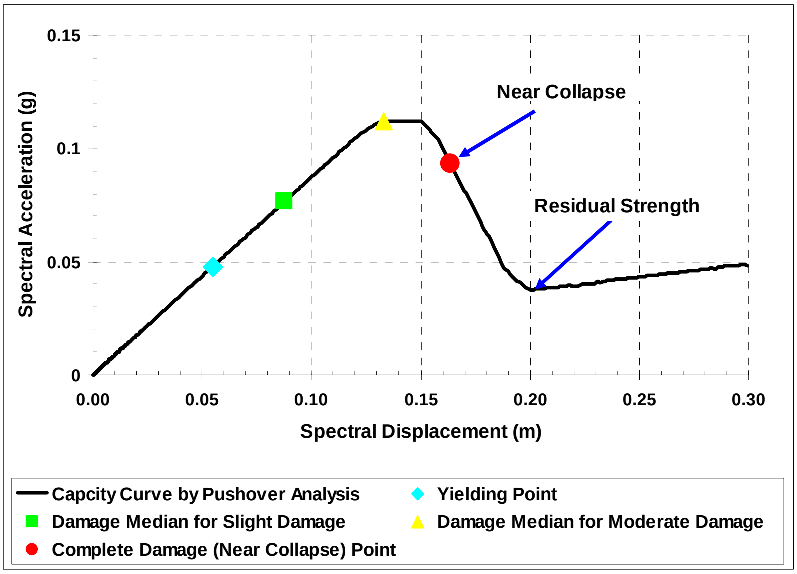
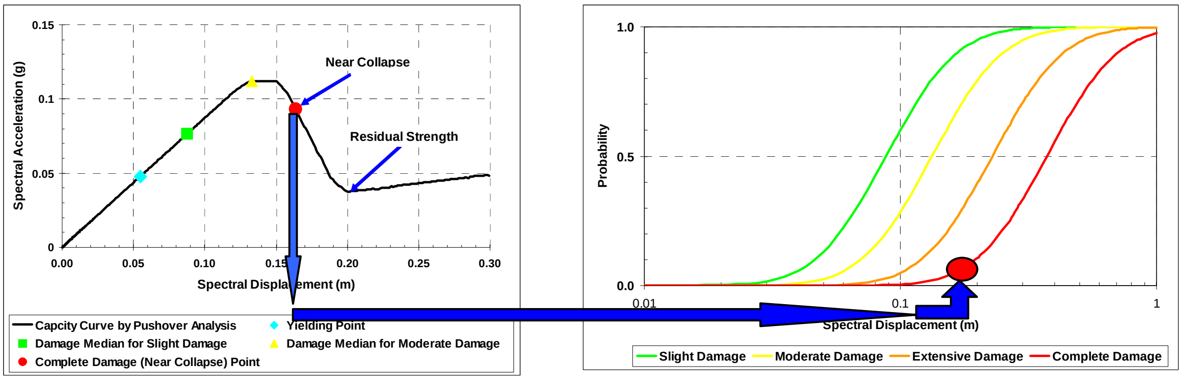


Fig. 12. The capacity spectrum from the nonlinear pushover analysis

To reduce the amount of calculations to a manageable level, a square grid, of the order of 100m x 100m, can be used and the GIS building database used to assign each building to a grid square by virtue of the central point of the building footprint. The building information within each grid square can then be summed to give the total floor area of each building type and height range. The population within each grid square is derived in the same way.



(a) Capacity spectrum (b) Fragility curves

Fig. 13. Determination of the fragility curve for the complete damage state

Input Ground Motions

The ground motion is defined in terms of demand spectra, which are the response spectra as shown in Figure 2 but re-plotted as spectral acceleration against spectral displacement. The ground motion spectrum at a particular site is that expected to occur at the base of the building structure and must take into account any site response effects and any other local site effects. From a Site Class zoning map (see Figure 1 for example) a GIS analysis can be used to assign a Site Class to each of the grid squares. To allow for variability of the site response effects, the median site response amplification factor together with the median plus and median minus one and two standard deviations need to be considered as shown in Figure 14. These ground motion spectra correspond to those shown for a Site Class D in Figure 2.

It can be seen that applying the procedure illustrated in Figure 6 the fraction of the building stock to experience Slight, Moderate, Extensive and Complete Damage can be determine for each of these 5 ground motions. An overall result is then derived by using a weighted sum of these values with a weighting of 0.4 used for the median value, 0.25 for the median plus and median minus one standard deviation and 0.05 for the median plus and median minus two standard deviation values. Allowances can also be made for the effects of liquefaction and steeply sloping ground.

Cost Calculation

To determine the cost due to seismic ground motion a Cost Ratio is applied to each Damage State in accordance with the HAZUS recommendations (FEMA, 2003). The Cost Ratios are 2%, 10%, 50% and 100% for Slight Damage, Moderate Damage, Extensive Damage and Complete Damage respectively. The overall damage Cost Ratio is a fraction of the replacement value of the building. The damage cost in each grid square is derived by multiplying the value of each building by its respective overall damage Cost Ratio and summing these for all buildings within that grid square.

While this procedure is valid for the building structure, non-structural elements of the buildings can also be assessed. HAZUS divides non-structural components into those that are drift-sensitive and those that are acceleration-sensitive. The relative construction value of the building structure, drift-sensitive non-structural components and acceleration-sensitive non-structural components needs to be considered.

For buildings in the Low-Code or Pre-Code category the HAZUS default parameters show that the damage to drift-sensitive non-structural components is always less than that experienced by the building structure. Considering the likely effects of repairs to the building structure to the non-structural components it was recommended that the damage Cost Ratio to non-structural components will never be less than the damage Cost Ratio to the building structure. It follows that the damage Cost Ratio to drift-sensitive non-structural components should always be the same as that derived for the building structure.

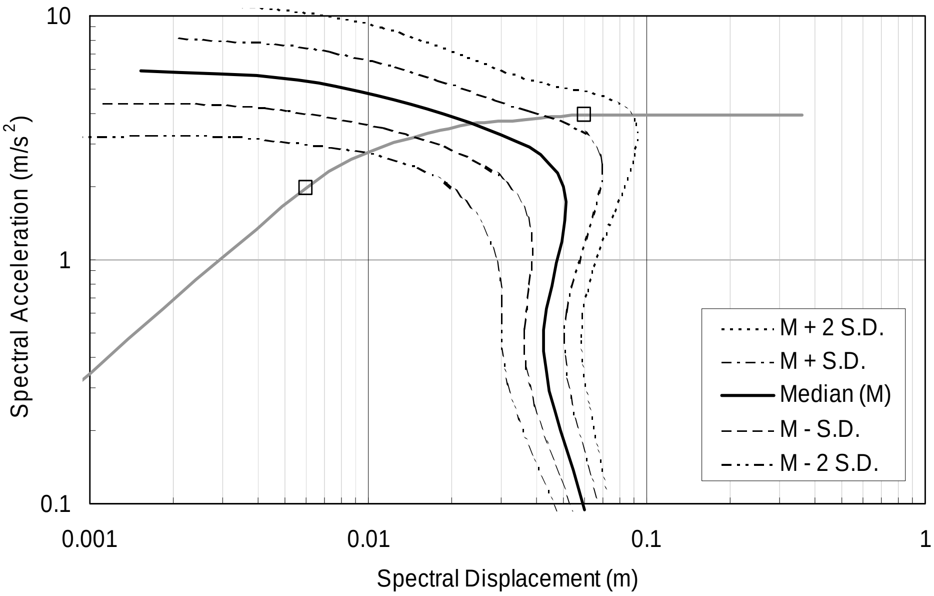


Fig.14. Example for the determination of spectral displacement for a HAZUS default pre-code low rise unreinforced masonry building when subjected to the 2% in 50 year uniform hazard ground motion for site class D shown in Figure 2

The damage Cost Ratio to acceleration-sensitive non-structural components is a function of the peak floor accelerations experienced by the structure and can vary quite significantly in comparison to the damage Cost Ratio to the building structure. Figure 15 shows an example calculation of damage Cost Ratio for low rise unreinforced masonry buildings defined using the default HAZUS coefficients. It can be seen that for the 10% in 50 year ground motion the damage Cost Ratios to both the building structure and the acceleration-sensitive non-structural components are similar. For the more frequent ground motion with a probability of being exceeded of 50% in 50 years however, the damage Cost Ratio to the non-structural components is greater than that to the building structure and must be considered directly in the overall cost calculation.

It must be noted that for low-rise and mid-rise buildings the peak floor accelerations will correspond very well to the spectral acceleration of the fundamental mode of the building. For higher buildings however where higher mode effects become significant the time-history analyses are likely to show that the calculated peak floor accelerations significantly exceed that implied by the spectral acceleration at the fundamental period of the building. Allowances, based on the results of the linear dynamic time history analyses, are likely to be required to determine the peak floor acceleration for high rise buildings.

Estimating Casualties

It is recommended that the HAZUS methodology be used as a basis to assess casualties arising from seismic ground motion. In this methodology four different Severity levels are assessed with Severity 1 being a minor injury, Severity 2 a serious injury requiring hospitalisation, Severity 3 a life threatening condition requiring intensive care and Severity 4 being instantly killed. The assessment of the quantity of casualties is derived as a percentage of the number of people in the building at the time of the earthquake and the level of damage sustained by the building.

The probabilities of injury severity level is recommended by HAZUS to be a function of the building Damage State and their recommended values for reinforced concrete frame and shear wall buildings are shown in Table 2. It can be seen that the probabilities of all injury severity levels are very dependent on the amount of building damage, particularly Collapse. It is noteworthy that there is a very low likelihood of critical injuries and death, Severities 3 and 4, except where there is Building

Collapse.

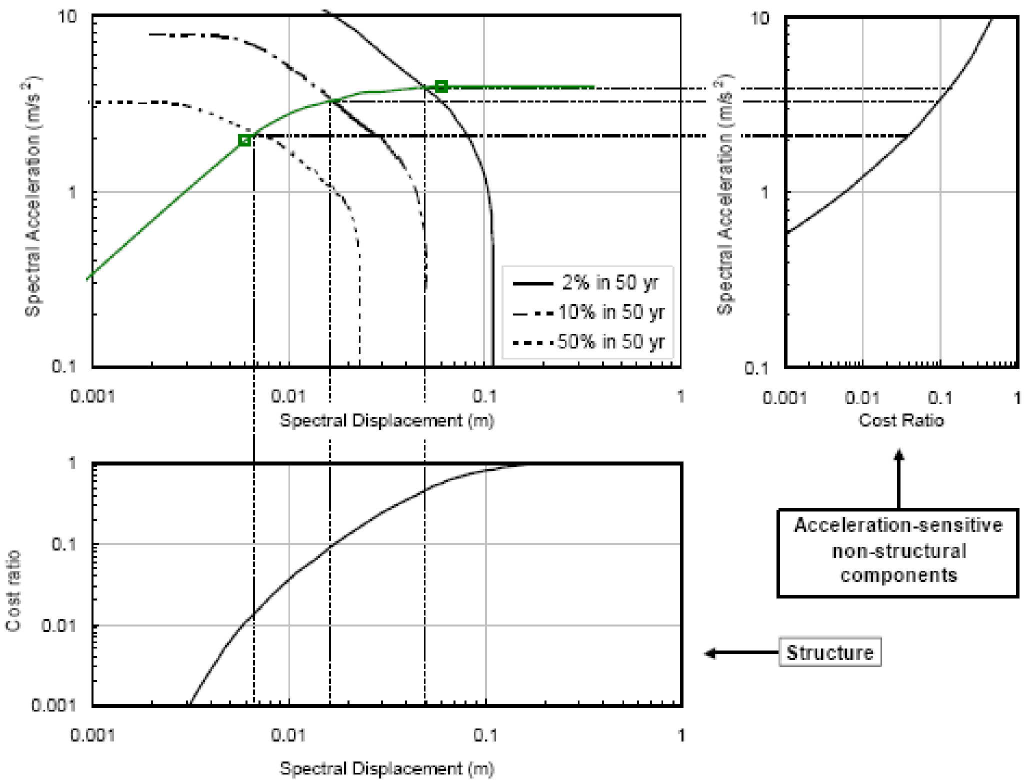


Fig. 15 Damage cost ratios for the structure and non-structural elements of a HAZUS default low-rise unreinforced masonry building subjected to three levels of ground motion

HAZUS recommends that the amount of Building Collapse is fraction of the buildings to experience Complete Damage. The number of casualties can be calculated for an earthquake ground motion occurring during day time (3:00pm) when the working population can be assumed to be at their place of work and at night time (3:00am) when everyone can be assumed to be at their residence.

Table 2: HAZUS Recommended Probabilities of Casualties as a Function of Building Damage State (from FEMA, 2003)

Injury Level	Building Damage State				
	Slight	Moderate	Extensive	Complete	Collapse
Severity 1	0.05	0.25	1	5	40
Severity 2	0	0.03	0.1	1	20
Severity 3	0	0	0.001	0.01	5
Severity 4	0	0	0.001	0.01	10

Estimating Annual Damage Cost

To determine the annual damage cost the predicted damage cost for each of the three ground motion levels (2%, 10% and 50% probabilities of being exceeded in a 50 year period) must be integrated over time. To do this calculation it is necessary to use the annual frequency that the ground motion is being exceeded rather than the probability of being exceeded within any given time period. The 2%, 10%

and 50% probabilities in 50 years correspond to annual frequencies of being exceeded of 0.0004, 0.0021 and 0.014. A smooth curve is fitted to these damage cost values and the curve then divided into small increments and the annual frequency of each increment determined. The annual frequency of each increment is determined by calculating the difference in the annual frequencies of being exceeded at each successive increment. The annual damage cost of each increment is then derived by multiplying the average damage cost for that increment by the annual frequency of that increment. The overall annual damage cost is the sum of the annual damage costs from all of these increments. It must be emphasized that the overall annual damage cost is the average over a very long period of time (several thousands of years) and is usually significantly influenced by the effects of very rare and quite large seismic events.

COST BENEFIT ANALYSES

Cost benefit analyses can be carried out to determine the economic benefit of introducing various code rules. The procedure basically comprises considering a range of buildings, and then, for each, determining what additional construction cost would be implied by the introduction of the code rules and determining the reduction in damage cost over the useful lifetime of the building. Only reinforced concrete frame buildings and reinforced concrete shear wall buildings will usually need to be considered as it is apparent that if a code is introduced unreinforced masonry buildings and buildings with unreinforced infill walls are not likely to be permitted.

When determining the additional cost the additional material cost of both reinforcement and concrete must be considered. Seismic design forces for the buildings can be derived from the soil surface spectra shown in Figure 2 using the procedure suggested by IBC 2003. The additional material cost of the detailing rules conforming to Intermediate Moment Frames as required by IBC 2003 must also be included. If a column size is required to increase the loss of floor area can also be assigned a cost. There will also be a cost due to additional construction time and design effort.

To determine the benefit from the estimated reduction in damage cost, the median values of the fragility functions can be adjusted as recommended by HAZUS. This amounts to changing from Pre-code Seismic Design values to those for Moderate-Code Seismic Design values. Figure 16 shows these values for the height ranges covered by the default HAZUS values. The risk calculations described above can then be repeated so that for each building type the annual damage cost for each Site Class is determined and compared to the original value. The benefit is then derived by multiplying the reduction in annual damage cost by the useful life of the building, often assumed to be 50 years.

CONCLUSIONS

The paper has presented a methodology for determining the seismic risk to buildings in a modern high-rise city in a region of moderate seismicity.

The input ground motions are determined for probabilities of being exceeded of 50%, 10% and 2% in 50 years using conventional seismic hazard calculation methods. The ground conditions need to be classified using the NEHRP recommendations and a Site Class zoning map produced. A methodology has then been presented for determining the amplification functions and their variability.

Ideally a GIS based building database is established to identify representative building types to enable a sensible classification system to be established. A method is presented for determining the building response to seismic ground motion that is largely based on the HAZUS methodology as described in FEMA (2003) and methods for calibrating this methodology to a new region and extrapolating it to high rise building types are discussed.

By combining the ground surface seismic motions with the building capacity curves and fragility functions the amount of building damage experienced by each building type on each soil Site Class is determined. A GIS analysis can then be carried out by laying a grid over the region to enable the total floor area of building damage to be determined for each level of seismic ground motion. At the same time the total damage cost and the number of casualties are estimated. A method for determining the annual damage cost from seismic loading (averaged over a very long time period) is presented.

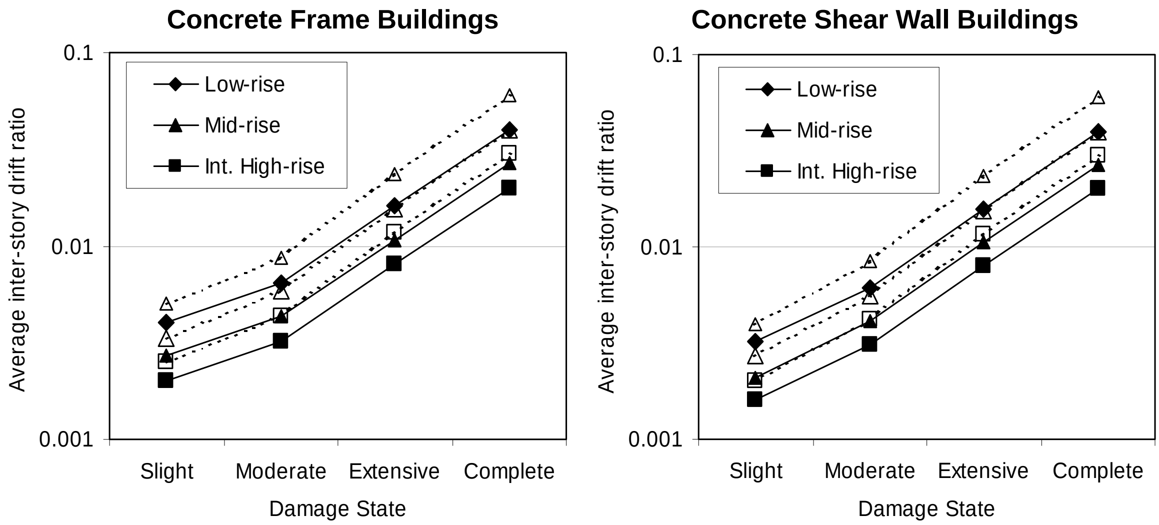


Fig. 16. Default Median Values of HAZUS Fragility Functions for Concrete Buildings; Solid Lines are for Pre-Code Seismic Design and Dashed Lines for Moderate Code Seismic Design

Finally a method to carry out a cost benefit analysis of introducing seismic design rules is discussed. The additional cost of applying a set of design rules is quantified in terms of additional construction materials. The benefit is derived by first deriving revised fragility functions assuming the buildings have been designed, detailed and constructed in accordance with the seismic design rules. The annual damage cost that would be expected if these rules were in place can then be determined. The benefit is the reduced annual damage cost applied over the service life of the building.

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