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SEISMIC FRAGILITY FUNCTIONS FOR TYPICAL URM SINGLE-STORY RESIDENTIAL STRUCTURES IN TRINIDAD AND TOBAGO

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ABSTRACT

Trinidad and Tobago is a developing twin island republic in the southern Caribbean. Preliminary seismic hazard analysis for the area indicates 0.2 second rock spectral acceleration values (S_s) on the order of 0.5g to 1.1g for significant parts of Trinidad, and up to 1.4g for south-west Tobago. At least two-thirds of the existing building stock comprises of single-story unreinforced masonry (URM) residential structures where the type of URM is of 100mm thick clay tile units, and the roof load is 0.3 to 0.5 kN/m^2. In order to enable probabilistic assessment of the structure before and after rehabilitation, or loss estimates of entire towns given certain scenarios, requires fragility functions of the structure for the various limits states describing damage. The IDA method, based on a series of nonlinear dynamic analyses of the structure, is implemented using the Zeus-NL computer program in order to obtain the response statistics. These results and the fragility analysis of the structure are presented in this paper culminating in the fragility functions for each limit state in terms of the S_a intensity measure. It is clear that this form of construction is particularly vulnerable therefore requiring rehabilitation as a high priority in order to mitigate likely substantial casualties and economic losses.

Introduction

Trinidad and Tobago is a developing twin island republic in the southern Caribbean. Preliminary seismic hazard analysis for the area indicates 0.2 second spectral acceleration values on the order of 0.5g to 1.1g for significant parts of Trinidad, and up to 1.4g for south-west Tobago. At least two-thirds of the existing building stock comprises of single-story unreinforced masonry (URM) residential structures where the type of URM is of 100mm thick clay tile, and the roof load is 0.3 to 0.5 kN/m^2. Therefore this situation is one of significant risk to the future development of Trinidad and Tobago. As a mitigation effort, a rehabilitation design for individual structures was completed in 1998 based on the use of overlays. However, recent technological developments, largely due to the performance-based design paradigm, enable probabilistic assessment of the structure before and after rehabilitation, or loss estimates of entire towns given certain scenarios, and hence can enhance the decision-making process of the various stakeholders in ways unavailable before.

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Such seismic probabilistic assessment however requires fragility functions of the structure. Studies have been conducted in the past for URM structures and included in fragility function libraries, such as for the ATC 58 loss estimation methodology (ATC 2007) currently being finalized. In this case, the fragility functions are incorporated into an overall loss estimation methodology conceptually given by (1),

$$v(DV)= \int\int\int G <DV \mid DM> dG <DM \mid EDP> dG <EDP \mid IM> d\lambda\,(IM) \qquad (1)$$

Equation (1) expresses the total probability theorem as conditional probabilities, each represented by an integral, which is a probability distribution function. The DV, DM, EDP, and IM refer to a decision variable, damage measure, engineering demand parameter, and intensity measure, respectively. The vulnerability, sometimes called the fragility as in this paper, is the integral with respect to the EDP|IM.

One of the more recent probabilistic and performance-based studies of URM buildings is by Wen et al (Wen 2004). Additionally, prior to the current probabilistic and performance-based paradigm, deterministic methodologies for the seismic evaluation of existing URM structures were developed for the collapse and life safety limit states. Brueau (Bruneau 1993, 1994) presented a comprehensive review of these methodologies that are all substantially influenced by the ABK methodology (Kariotis 1984), including the ASCE 31 (Hom 2003), which is one of the most recent.

These probabilistic and deterministic methods are valuable, but the form of URM construction used in Trinidad and Tobago is quite different from other typical URM structures used worldwide. It is based on more slender load-bearing walls without stiffening in the out-of-plane direction by cross-walls because of inadequate wall-to-wall connections. Furthermore, the low bearing stresses preclude the favorable rocking response. These factors suggest that the structure may be more vulnerable than other more typical forms of URM construction used worldwide.

This paper presents the results of fragility analyses of the structure and the resulting fragility functions. Central to the fragility analysis is the use of the IDA method based on nonlinear dynamic analysis of the structure.

Structural Model

The typical form of construction of the single-story residential structures is of walls comprised of clay tile masonry units for both the load-bearing and the internal partition walls. These units are manufactured in accordance with ASTM C112 and used with the cells in the horizontal orientation. The clay tile unit is of dimensions 200 mm high by 300 mm long by 100 mm thick, with web and shell thicknesses of approximately 8 mm. The unit weighs 5.5 kg, and has a compressive strength (average of 5 units), of approximately 3.5 MPa.

Fig. 1 shows the layout of the typical residential structure. It is of rectangular plan, 9.0 m wide by 11.0 m long and the story height from ground level to the top of the walls is 2.4 m. The roof is of gable shape (though sometimes hipped) of slope 22 to 30 degrees from the horizontal. It is comprised of galvanized steel corrugated sheeting supported by 50 mm by 100 mm timber secondary beams, or 100 mm cold-steel Z-purlins. In either case the spacing of these beams is approximately 1.0 m.

These secondary beams are supported by 150 mm deep by 50 mm timber main beams (rafters) at a spacing of 0.6 m to 1.2 m, or rolled I-section structural beams of 100 mm or 150 mm depth at a spacing dependant on the beam used. The main beams are supported by the wall by directly bearing on it (i.e. hangers are not used). If the main beam is of timber, at the top of the wall is a 100 mm by 100 mm timber element to which the beam is attached via one of a number of methods including bent-over steel rebar, or hurricane straps. If the main beam is of structural steel, various methods are used for connection to the wall including short steel columns. In the former case, the 100 mm by 100 mm timber element is bolted to a reinforced concrete (RC) beam that is cast over the masonry wall below. "Blocking" between the main beams, thus forming a continuous chord on the edge of the roof along the load-bearing walls, is not used.

In the case of the steel beam and short column, the latter is bolted to or anchored in the RC beam. The RC beam is 300 mm deep by 125 mm wide and contains 2 no. 12 mm mild steel rebar at top and bottom, and 6 mm mild steel 2-leg closed stirrups at 250 mm spacing. The RC beam is called a "ring beam" or a "belt beam" since it connects the perimeter walls. The perimeter walls in the long-direction are gravity load-bearing, but in the other direction above the ring beam the space is enclosed using the same clay tile masonry units, thus making a triangular shape in the gable-end, and without anchorage to the ring beam. The total roof weight is typically in the range 0.3 to 0.5 kN/m^2 and the wall weighs approximately 3.9 kN/m.

In firm soils the foundation for the perimeter walls consists of RC wall footings 0.6 m wide by 0.25 m deep with 150 mm thick reinforced masonry forming a short stem. In soft soils, 4.0 m long and 300 mm diameter bored RC piles are used at a3.0 m spacing. The internal flooring is a RC slab-on-grade 100 mm thick and reinforced with fabric reinforcement, typically 142 mm^2/m. The slab areas below internal partition walls are often thickened. For all concrete work the concrete is typically of a 28-day compressive strength of 21 MPa. The internal partition walls are connected to the load-bearing external walls by the practice of "toothing" the former into the latter, and not by laying-up the walls simultaneously. At the corners of the perimeter walls, sometimes the walls are tied using 6 mm mild steel rods in the mortar joints, at various vertical spacings.

The lateral load resisting structural system of the structure as described above can therefore be classified as a box system of unreinforced masonry (URM) shear walls. URM shear wall structures are acknowledged to have four fundamental possible modes of response in the in-plane direction– flexural leading to toe compression failure, shear leading to diagonal tension failure, rocking, and sliding. The actual response is frequently a combination of these modes and the level of bearing stress on the wall is a very significant factor determining which mode will dominate the response. In the out-of-plane direction, after formation of a horizontal through-wall crack at the base, depending on the level of bearing stress, a stable rocking response is possible. However, there is a level of lateral displacement beyond which failure by dynamic instability will likely occur.

Given the description of the structure, and considering test data on the in-plane response of a prototype of the wall (Clarke 1998), the following presumptions regarding its behavior hence structural modeling are made:

1. Under significant lateral load the "toothed" connection of the internal to external walls will cause a vertical line of weakness in the external walls and separate them into a set of vertical elements, from ground level to the top of the wall, interconnected at the top by

the ring beam. Pier regions at the sides of openings in the walls are also modeled in this manner since it is typically the case that one vertical edge of a pier coincides with an internal partition.

2. The bearing stress on any wall element, and its self-weight, are sufficiently low that any element loaded in-plane will respond in the sliding mode only.
3. The sliding in-plane load-displacement response is nonlinear and assumed to be of elastic-perfectly plastic form.
4. A wall element deforms linearly and elastically before and during sliding.

For maximum efficiency using a 3-dimsional analysis, a "fiber model" form of the finite element method (FEM) can be used for the structural modeling of the house for dynamic analysis. In such a model, specific element types are considered (e.g. beams, columns, etc), the constituents of a section of an element are defined, and the mechanical properties determined by integrating over the section. The Zeus-NL computer program for the inelastic dynamic analysis of structures by Elnashi et al of the Mid-America Earthquake Center (Elnashi 2009), was used. In Zeus-NL, material nonlinearity is implemented by using the nonlinear stress-strain relations of possible constituents, and joint elements are available which further enable simulation of nonlinear behavior at interfaces.

Therefore in this study, each wall element was modeled in Zeus-NL as an elastic column, and at each wall-to-support interface, a joint element was used to simulate the nonlinear sliding, given point 3 above. The clay unit wall modulus was taken as 3400 MPa based on the recommendations of ASCE 41-06 (ASCE 2007). A solid wall section was assumed rather than determining an equivalent but smaller section to account for the voids. For the joint element, the stress to initiate sliding was taken as 0.13 MPa, with an initial stiffness of 10.7 kN/mm reducing to an arbitrarily small amount thereafter. These values correspond to those for a 2.4 m long wall and are therefore conservatively used for all the walls, in any translational direction. With respect to rotational degrees of freedom at a joint element, these are given arbitrarily large stiffness values.

Although the roof is flexible, as its connection to a wall lacks a continuous chord to which the main beams are well anchored, it cannot act like a diaphragm. Its effect on the supporting wall is therefore merely to provide a bearing load without pushing under the earthquake load by flexural action in the out-of-plane direction. To account for the distribution of the inertial forces on the structure in the dynamic analysis under ground motion, a mass was lumped at the top of each element, coinciding with its top node, and of value determined by considering the tributary roof weight, the wall element's weight, and the tributary lengths of the of RC ring beam at both sides of the element. Since a typical wall element is connected to a joint element, in turn connected to the support node, the latter is modeled as fixed with respect to translation and rotation about the 3 axes. Fig. 2 shows the essential elements of the physical model of the structure. Note that the corner column in the porch area is omitted as it is structurally insignificant under an earthquake. Figs. 3 and 4 are of the structural model in Zeus-NL, and the modeling of a typical wall element, respectively.

Ground Motion Input

The recording of strong ground motions due to earthquakes in the Caribbean is in its infancy therefore ground acceleration records required as input to the dynamic analysis are not readily available. Ten ground motion far-field records from the PEER Strong Motion Database (PEER 2009) were arbitrarily selected and their characteristics are shown in Table 1. Each of these was used to derive an artificial accelerogram that is compatible with the IBC 2006 design acceleration response spectrum for Site Class D, and using the (interim) S_s and S_1 2%/50-year maps of the Seismic Research Center of The University of the West Indies, for a site located in the capital city of Port-of-Spain. At the zero-period point in the design accelerogram, the spectral acceleration equals the PGA and is approximately 0.28g. The artificial accelerograms were calculated using the Kumar algorithm and Spec3 software (Kumar 2006). An advantage of this approach is that the resulting accelerogram has the same Fourier phases as the PEER records, hence the same (relative) damage potential.

Damage Measure

The story drift ratio was selected as the damage measure (DM) for the in-plane response of the wall elements. The four limit states defined by HAZUS-MH MR4 (DHS 2003) for the "low-code" case were used to monitor this damage. These are: complete damage (CD) – 3.5%, extensive damage (ED) – 1.5%, moderate damage (MD) – 0.5%, and slight damage (SD) – 0.3%. In the out-of-plane direction, the occurrence of dynamic instability (DI) was monitored by comparing the displacement with the displacement capacity, x_c, derived by Priestley (Priestley 1985) using an energy approach.

$$x_c = b(P + W)/ [\, 2P + W] \qquad (2)$$

where b is the wall thickness, and P and W are the bearing load and self-weight on the wall element, respectively.

Fragility Analysis

Fragility is the probability of exceeding a limit state as a function of an intensity measure (IM) of the ground shaking. The spectral acceleration, S_a, was selected as the IM as it is thought to require a minimum of ground motion records for the same confidence level (Shome 1998). Furthermore, the resulting functions can then be used in probabilistic frameworks that are expected to be adopted for developing risk analysis tools for Caribbean application.

The fragility function can be represented by the 2-parameter lognormal CDF, Φ (Cornell 2002). The 2 parameters are the mean of the ln (DM), λ, and the standard deviation, ξ, of the ln (DM). Hence if D is the limit state corresponding to a DM, then

$$\text{Probability of exceeding a limit state} = P\,(\leq D) = \Phi\,[(\ln\,(IM) - \lambda)\,/\,\xi] \qquad (3)$$

In this study, only uncertainty due to the ground motion is considered and not uncertainty due to material properties, etc. To obtain the data points required for the statistical analysis, the Z-Beer utility of Zeus-NL was used to perform incremental dynamic analysis (IDA) or dynamic pushover analysis (Vamvatsikos 2002). By also using Zeus-NL, an eigenvalue analysis of the structure indicated a first mode period of 0.785 sec. This relatively high value for a single-story structure is due to the flexibility of the walls given the thinness of the webs and shells of the clay tile unit. From the design spectrum, this implies a S_a of 4.479 m/s^2.

Since the ground motion records are spectrum-compatible, this corresponds to a scale factor (SF) of unity. The ten accelerograms are applied to each orthogonal direction of the structure hence providing 20 data points per limit state. The accelerograms are scaled from an SF of 0.3, to an SF of 7.2, in increments of 0.3. Therefore the S_a range from 0.138g to 3.287g and there are 24 points per pushover curve which are enough to enable sufficiently accurate interpolation. A wall element is selected in each orthogonal direction to represent the structure's in-plane performance. This is also done for the out-of-plane response. The lower of the two S_a values for each run is then used for each of the four lognormal functions of the four in-plane limit states, and for the limit state of dynamic instability in the out-of-plane direction.

Results and Discussion

Fig. 5 shows the five fragility curves for the single-story URM structure. Table 2 shows the parameter values of each fragility curve.

Though not presented in this paper, in general the shorter direction of the house is more susceptible to damage, which is typical. However, as is observed in Fig. 5 for a given $S_{a,}$ the exceedence probability is higher for the DI limit state than for the ED and CD limit states indicating that the structure will likely collapse in this mode before the walls experience substantial damage in the in-plane direction. The physical reason for this occurrence is that as the walls that fail in the out-of-plane direction are very slender and flexible, and have no cross-walls to stiffen the response, they more easily reach the limit of dynamic stability before the in-plane walls, to which they transfer shear via the "ring beam", can respond significantly.

When this occurs, it is likely that the wall elements in the out-of-plane direction will physically disengage from the structure and fall out. On going from near the middle of the long direction of the structure towards the in-plane walls, the inner elements displace more and will fall out first. However, all the elements on those sides also support the roof and the "ring beam" so the roof, hence entire structure, will likely physically collapse subsequent to the progressive fallout of the out-of-plane walls due to dynamic instability.

This occurrence is unlikely for other more commonplace forms of URM construction because, assuming sufficient cross-walls to limit the pushing of the flexible diaphragm, these cross-walls also remain sufficiently effective in stiffening the out-of-plane response, that significant damage to the structure, and risk of collapse, is more likely for the in-plane walls. The out-of-pane walls are then only susceptible to the lateral vibration of its distributed mass as a two-way spanning panel, depending of the edge conditions. This mode of response is simply not possible with the Trinidad and Tobago construction and manifests as indicated in Fig. 5. The DI limit state dominates the fragility of the structure to such an extent, that the CD in-plane response limit state remains unlikely. Interestingly, the S_a values for the ED and CD limit states (if the DI limit could be prevented from occurring) are relatively high. This is due to the principal response mechanism of sliding for the in-plane walls, which is a form of base isolation.

If the HAZUS fragility curves for URM construction are overlaid (not shown) on those of Fig. 5, it will be clear that this URM structural system is a special case particularly in need of rehabilitation. Given its prevalence in Trinidad and Tobago, this represents a significant threat of large scale economic loss to a developing country which also plays a major role in the economics of the Caribbean as a whole.

Though the assumptions made for the structural modeling are expected to yield conservative results, before application to practical risk or loss assessment it is generally necessary to determine the confidence bounds on the calculated fragility curves due to the epistemic or knowledge-based uncertainty such as that due to the structural modeling. Various methods exist for determining these bounds and given the intended scope of the functions presented in this paper for Caribbean application, a rigorous approach is to be used again involving nonlinear dynamic analysis and IDA. This is the subject of future work.

Conclusions

The conclusions of this study on the fragility of typical single-story URM residential structures in Trinidad and Tobago are as follows:

1. The analytically derived lognormal fragility curves and their parameters are as presented in Fig. 5 and Table 2, respectively.
2. The curves indicate a particular susceptibility to failure by dynamic instability in the out-of-plane direction due to the high flexibility and slenderness of the walls, and the low bearing stress imparted by the roof structure and wall self-weight.
3. Given its prevalence in Trinidad and Tobago, this represents a significant threat of large scale economic loss to a developing country which also plays a major role in the economics of the Caribbean as a whole.
4. The fragility curves presented herein can be used as part of seismic risk assessment of the Caribbean to provide justification of rehabilitation proposals.

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Figures and Tables

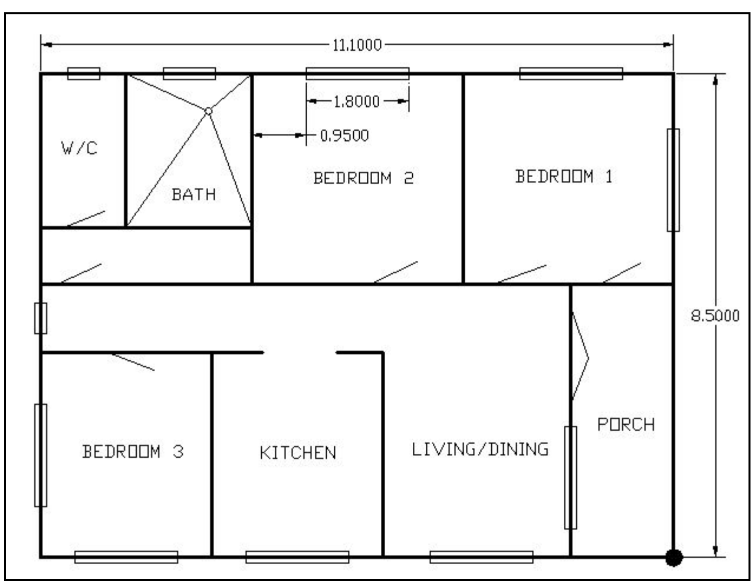

Figure 1 Structure Layout

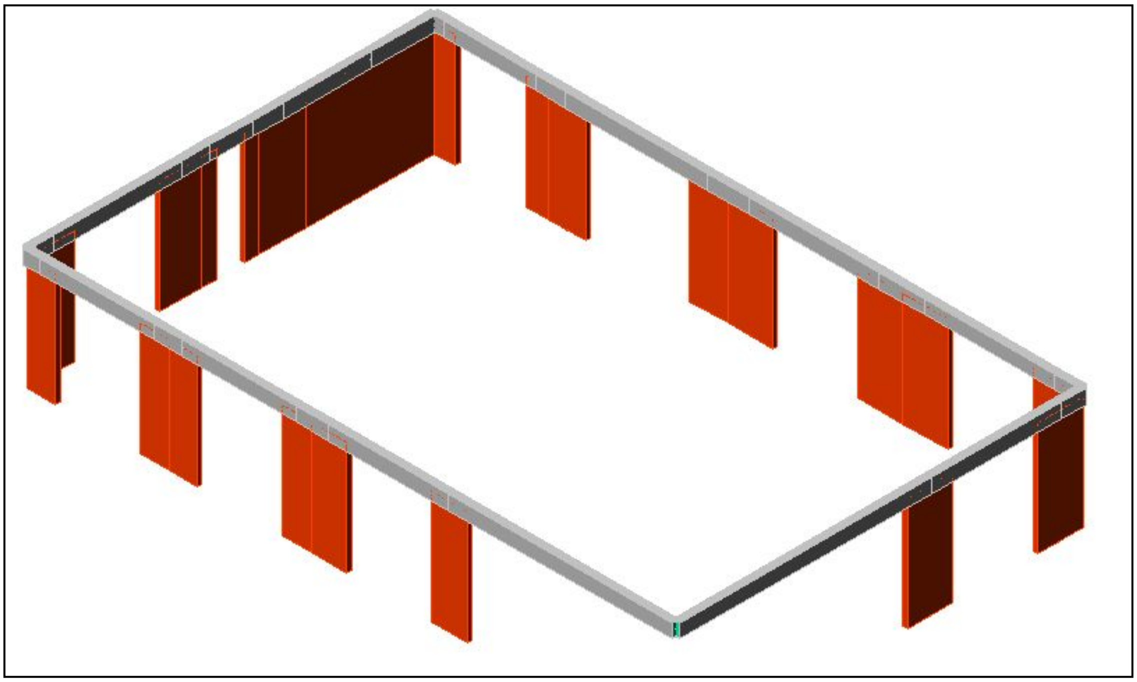

Figure 2 Main Structural Elements

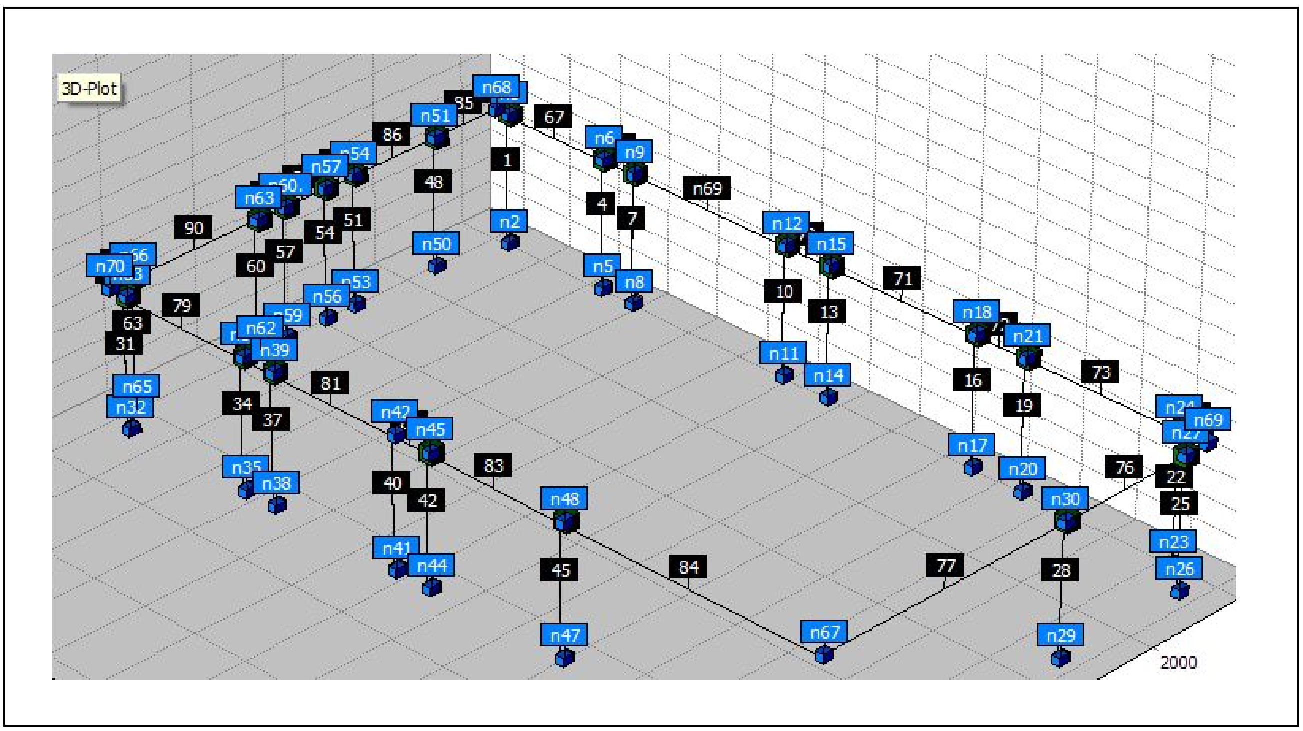

Figure 3 Zeus Model of Structure

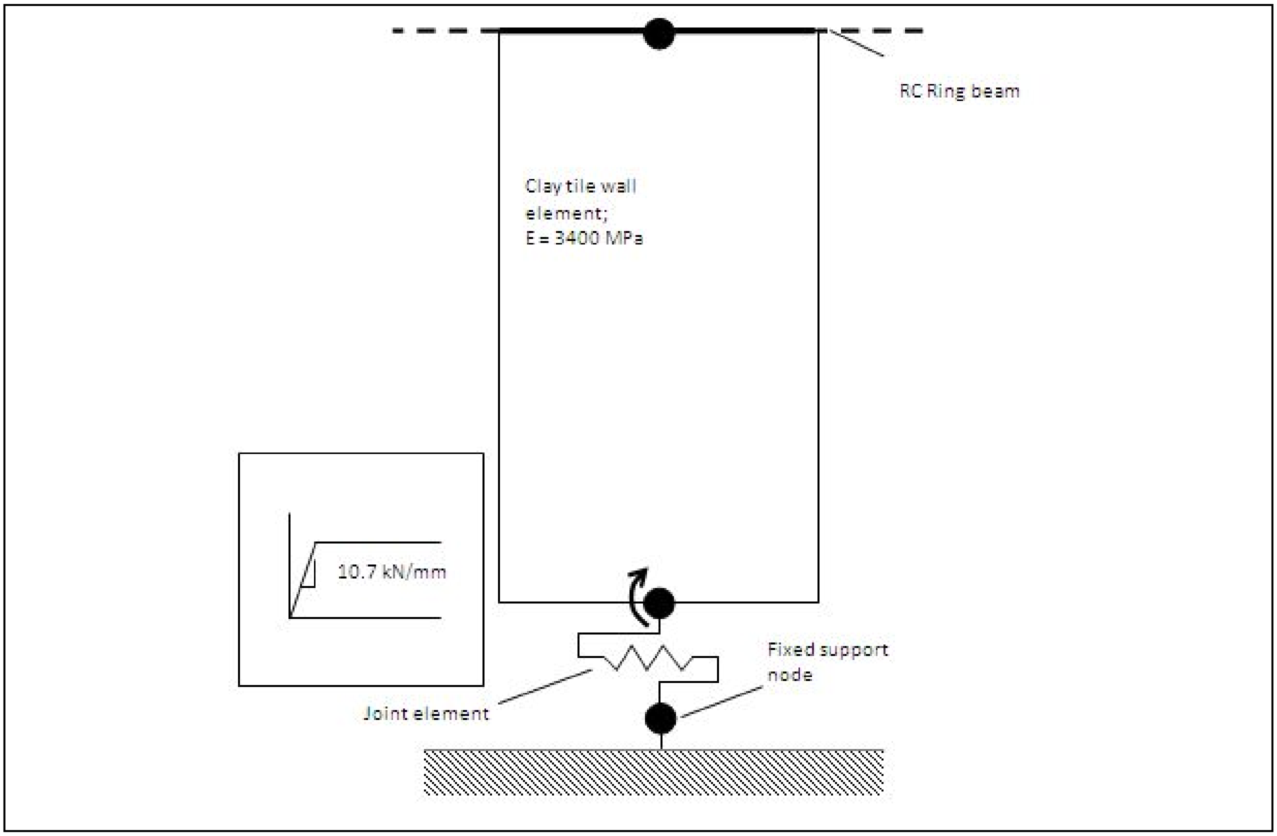

Figure 4 Structural Model of Wall Element

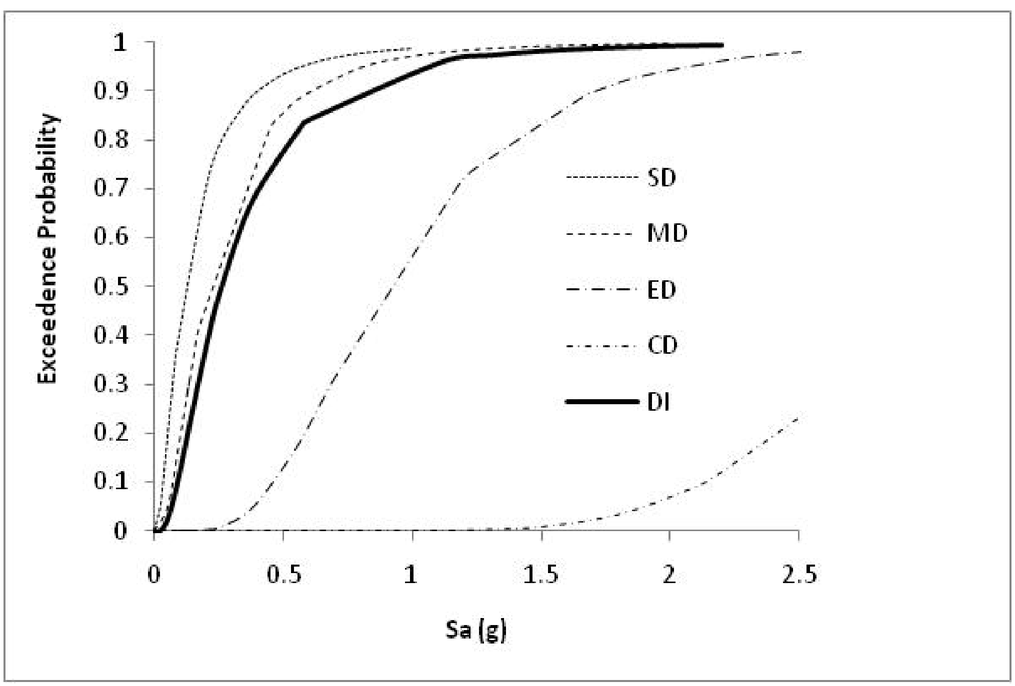

Figure 5 Fragility Curves

Table 1. Earthquakes for Ground Records

Earthquake File	Event Name	Magnitude	PGA (g)	Distance (km)
CPE147	Imperial Valley 79	6.6	0.169	26.5
CPE237	Imperial Valley 79	6.9	0.157	26.5
DSP000	Landers 92	4.4	0.171	23.2
DSP090	Landers 92	7.4	0.154	23.2
JOS000	Landers 92	7.4	0.274	11.6
JOS090	Landers 92	7.4	0.284	11.6
MV000	Landers 92	7.4	0.188	19.3
MV090	Landers 92	6.5	0.140	19.3
NPS000	Landers 92	7.3	0.136	24.2
PT315	Imperial Valley 79	7.4	0.204	14.2

Table 2 Parameters of Lognornal Fragility Curves

Limit State	$\mu_{\ln Sa}$	$\sigma_{\ln Sa}$
Minor Damage	-2.14215	0.954529
Moderate Damage	-1.56825	0.829976
Extensive Damage	-0.11028	0.50836
Complete Damage	1.147436	0.303178
Dynamic Instability	-1.34405	0.826642

References

ASCE, 2007. *ASCE 41-06: Seismic Rehabilitation of Existing Buildings*. ASCE.

Applied Technology Council, 2007. *Guidelines for Seismic Performance Assessment of Buildings, ATC-5835% Draft.*

Bruneau, Michel, 1994. State-of-the-Art Report on Seismic Performance of Unreinforced Masonry Buildings. *Journal Structural Engineering* ASCE. 120(1), 230-251.

Bruneau, Michel, 1993. Seismic Evaluation of Unreinforced Masonry Buildings – A State-of-the-Art Report. *Canadian Journal of Civil Engineering*, 21, 512-539.

Clarke, R. P, 1998. *The Hysteretic Behavior of Ferrocement-Retrofitted Clay Block Masonry Walls Under In-Plane Reversed Cyclic Lateral Loads: Experimental Investigations and Graphical Computer Models*. PhD Thesis. The University of the West Indies, St. Augustine Campus, Trinidad.

Cornell, A.C, Jalayer, F., Hamburger, R.O., and Foutch, D.A., 2002. Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *Journal Structural Engineering,* ASCE, 128(4)526-533.

Department of Homeland Security, 2003. *HAZUS MR4 Technical Manual. Multi-Hazard Loss Estimation Methodology*. Department of Homeland Security.

Elnashi, A.S, Papanikolaou, V.K, and Lee, D. H, 2009. *Zeus-NL A System for Inelastic Analysis of Structures, Version 1.8.7*. Mid-America Earthquake Center, University of Illinois at Urbana-Champaign.

Hom, D.B, Poland, and C.D, 2003. *ASCE 31-03: Seismic Evaluation of Existing Buildings*. ASCE.

Kariotis, J.C, 1984. Survey of Unreinforced Masonry Buildings. *Reconnaissance Report of the Coalinga, California Earthquake of May 2, 1983*. Earthquake Engineering Research Institute, El Cerrito, California, pp. 145-156.

Kumar, Ashok, 2006. Software for Generation of Spectrum Compatible Time History Having Same Phase as of a Given Time History. *Proceedings of the 8th U.S. National Conference on Earthquake Engineering*, April 18-22, San Francisco, California.

PEER Strong Motion Database. Pacific Earthquake Engineering Research Center, Berkeley, California. http://peer.berkeley.edu/smcat/ (16 Nov 2009).

Priestley, M.J.N, 1985. Seismic Behaviour of Unreinforced Masonry Walls. *Bull. N.Z Soc. Earthquake Engineering*, 18(2), 191-205.

Shome, N, Cornell, A. C, Bazzurro, and Carballo, J.E., 1998. Earthquake, Records, and Nonlinear Responses. *Earthquake Spectra*. 14(3). 469-500.

Vamvatsikos, D, and Cornell, A.C.,2002. Incremental Dynamic Analysis. *Journal of Earthquake Engineering and Structural Dynamics*, 31, 491-514.

Wen, Y.K, Ellingwood, B.R, and Bracci, J., 2004. *Vulnerability Function Framework for Consequence-Based Engineering*. MAE Center Project DS-4 Report. Mid-America Earthquake Center, University of Illinois at Urbana-Champaign.