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SEISMIC PROBABILISTIC RISK ASSESSMENT OF CMU WALLS IN NUCLEAR POWER PLANTS USING A NONLINEAR MODELING STRATEGY

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Abstract

Advances in seismology have led to the perception that the potential earthquake hazard in the United States may be higher than originally assumed. In recent years, the nuclear industry and the Nuclear Regulatory Commission have made a tremendous effort to assess the safety of nuclear power plants. The Seismic Probabilistic Risk Assessment (S-PRA) is used to upgrade the nuclear power plants in the U.S. to comply with the new seismic hazard. By performing a S-PRA, the dominant contributors to seismic risk and core damage frequency can be identified. The assessment of component fragilities is a crucial task in S-PRA from which a High Confidence of Low Probability of Failure (HCLPF) capacity can be calculated. Concrete Masonry Unit (CMU) walls are historically known to be a weak link that controls components HCLPF capacities through seismic interaction, potentially leading to higher core damage frequencies.

The structural assessment of CMU walls poses significant challenges due to the need for modelling complex nonlinear material behavior. Hence, current practice in the nuclear industry relies on simplified finite element analysis using elastic isotropic shell elements to analyze CMU walls. While these models provide insight into the elastic performance of CMU walls, they tend to be conservative since damage evolution, failure modes, deformation capacity and out-of-plane strength are not properly accounted for in the analysis.

This paper presents analysis that takes into account the nonlinearity of materials to calculate the HCLPF capacity of a CMU wall. Nonlinearity was incorporated through a resistance function and nonlinear springs, in an equivalent frame model, which was developed in the commercial software SAP2000. The Conservative Deterministic Failure Margin methodology was used to calculate a HCLPF capacity. For comparison, a HCLPF capacity was also calculated considering elastic behavior of the CMU wall.

Keywords: CMU Wall; Risk Assessment; Nonlinear Modeling; Nuclear Power Plants; Seismic Evaluation

1. Introduction

Concrete Masonry Unit (CMU) walls in nuclear power plants (NPPs) are historically known to be one of the weak links that controls component High Confidence of Low Probability of Failure (HCLPF) through seismic interaction, potentially leading to higher core damage frequencies. The structural assessment of CMU's poses significant challenges due to the need for modeling complex nonlinear material behavior. However, in an effort to conduct reliable nonlinear seismic analysis of masonry buildings, several simplified numerical methods have been proposed in the last twenty years [1], and most of them have been validated through experimental data. These modeling strategies can represent, with certain degree of accuracy, the strength, stiffness, and deformation capacity of masonry walls. Also, they can account for the combination of highly nonlinear materials, complex geometric configuration and static and dynamic loading.

Nonetheless, despite of these new advances, the current practice in the nuclear industry relies on simplified finite element analysis using elastic isotropic shell elements and simple beam elements to analyze CMU walls as noted in [2]. While these models provide insight into the CMU's performance in the elastic range, their applicability is quite conservative since damage evolution, failure modes, deformation capacity and out-of-plane strength cannot be accurately predicted.

This paper attempts to fill this gap by discussing the state-of-the-art modeling strategies available in the literature for the seismic assessment of CMU walls in nuclear power plants subjected to out-of-plane loading. A typical hierarchy of analyses approaches including macro modeling, homogenization techniques and micro modeling, suitable for the nuclear industry are discussed.

The paper also presents a case study to gain insight on how the macro modeling techniques could realistically capture the structural behavior of the walls compared to the standard practice in the attempt to achieve realistic HCLPF values. For the case study, three HCLPF capacities were calculated for out-of-plane masonry walls. The first, taking into account elastic parameters and results with the methodology proposed in [2]. The second value was calculated with results obtained from the resistance function from [3]. The third value was calculated with the nonlinear parameters and results obtained from an analysis considering the nonlinearity of the masonry derived from the use of macroelement models as a solution to the modeling strategy to use. The geometric properties of the studied wall are obtained from [3]. The HCLPF will be defined with respect to the Peak Ground Acceleration (PGA) of the selected earthquake.

2. Modeling Strategy

2.1 Overall Overview of Modeling Strategies for Out-of-Plane Analysis of Masonry Walls

Numerical modeling of structural masonry is one of the most complicated problems in structural engineering research and practice [4]. Modeling strategies can represent, with certain degree of accuracy, the strength, stiffness, and deformation capacity of piers and spandrels [5]. A typical hierarchy of analysis approaches includes Block-Based Models (BBMs), Continuum Models (CMs) and Macroelement Models (MMs) which are suitable for the nuclear industry are discussed next.

The BBMs attempt to simulate the response of the material at the scale of the principal heterogeneity of masonry, which is comprised of blocks assembled with mortar joints. One of the principal favorable characteristics of BBMs is the direct description of the actual masonry layout and of the principal overall mechanical properties of block, mortar and the bond between them but this leads to massive computational effort. This limitation typically restricts the application of these numerical approach to small masonry panels [5].

CMs are based on a continuum deformable body to simulate masonry walls. Because of the mesh size could be considerably greater than the block size, the computational demand of these numerical strategies is usually lower than BBMs. Nevertheless, the formulation of adequate homogeneous constitutive laws for masonry is a challenging task even when given the mechanical features of masonry [6].

MMs represent the structure as an arrangement of panel-scale structural components with a mechanical-based behavior. The attractive features of these models (e.g., limited computational demand, easy arrangement of the model discretization and simple definition of the mechanical properties) have led to their widespread utilization [1]. However, MMs typically assume there are no local failure modes and this could lead to not taking into account in the proper way some important effects such as the out-of-plane damage due to the in-plane damage, and vice versa [7].

There are several modeling strategies that attempt to capture behavior of masonry walls due to seismic loads [8-16]. MMs are commonly the preferred approach for the seismic assessment of masonry structures [17-22]. Their relatively small computational effort, coupled with the easy and quick definition of the model and mechanical properties, make them suitable for the purpose of this study.

2.2 Selected Modeling Strategy

The modeling technique for the out-of-plane CMU selected herein is the Equivalent Frame Method (EFM) [23]. This is a widely used strategy and can be implemented for pushover analysis [24]. Because its computational cost and analysis time are low compared to a finite element model, this methodology is typically preferred for performance-based analysis when material non-linearity is considered.

Masonry piers, which are the vertical elements, can be replaced by beam elements. These beams have dimensions H, L and t. The modulus of elasticity of the masonry is designated as Em. Also, each beam has a dominant failure mode, at mid-height of the wall [3], for which a force-displacement curve can be determined in advance using equations given by [3].

As discussed before, there are also some disadvantages that must be taken into account when using this strategy. Macroelement models usually assume that any activation of local failure mode is avoided. This decoupling assumption could lead to a conservative estimate of the seismic capacity, as, in reality, out-of-plane and in-plane damages can simultaneously arise [22]. This was resolved using the function for out-of-plane failure modes from [3].

There are some additional limitations when applying this methodology. Primarily, SAP2000 [25] cannot account for the axial force-bending moment interaction when using nonlinear elements (designated Link elements in SAP2000) to simulate rocking or sliding behavior. The ultimate failure mode of the pier is affected by the axial force distribution in the piers and modifies significantly the response as the wall enters in the nonlinear range of material [4]. Furthermore, this methodology does not differentiate between joints and masonry units. Generally, most available MMs studies investigate the elastic linear behavior of brickwork, but the inelastic behavior of masonry structure has been studied less frequently. This is because the MMs methodology requires several mechanical parameters to be defined, and the analysis of different masonry layouts typically need multiple calibrations calibration of model parameters [26].

3. Case Study

This section presents a case study to better understand how the macro modeling techniques for CMU walls can achieve more realistic HCLPF values. Two HCLPF capacities were calculated for out-of-plane masonry walls. The first value was calculated using elastic parameters and results with the methodology proposed in [2]. A second value was estimated considering the nonlinear parameters and results obtained from a pushover analysis carried out by the authors.

3.1 Model Description

The non-load bearing reinforced CMU wall (M-00 from [2]),]), shown in Fig 1 (a) [3], was modeled using SAP2000 software. The height, length, and thickness of the wall are 78.74 in (2 m), 55.906 in (1.42 m) and 3.543 in (0.09 m), respectively. The wall has a vertical reinforcement ratio of 0.61%, as shown in Fig 1 (b). The base and the top of the wall are considered fixed and pinned, respectively.

Material properties are provided in Table 1. The masonry is considered to have a weight of 0.081 lb/in^3.

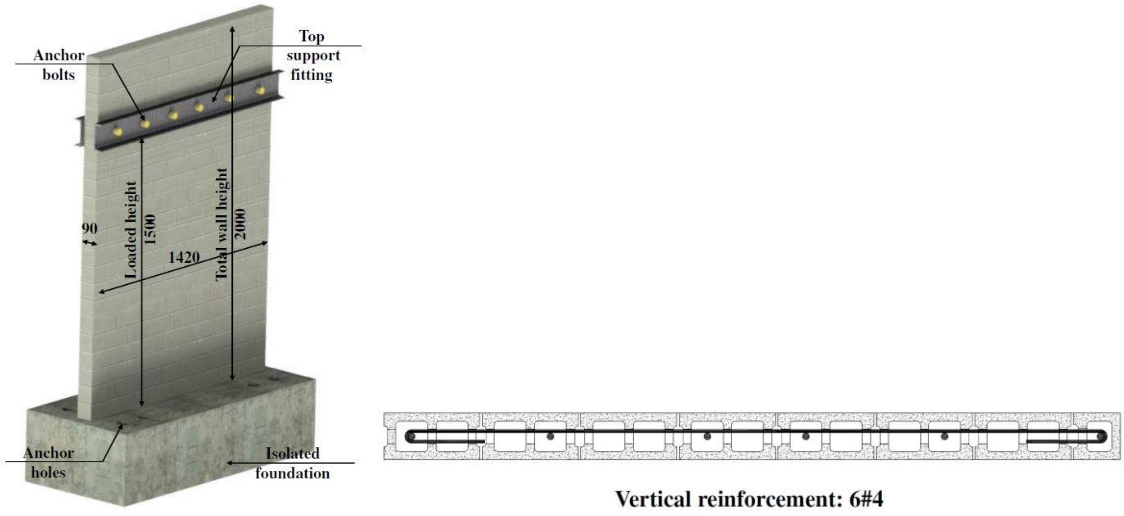


Fig. 1 – Studied Wall: (a) 3D View for the Wall (in Millimeters) (b) Typical Cross Section [3]

Table 1 – Material Properties

Material	**Modulus of Elasticity (ksi)**	**Yield Strength (psi)**	**Compressive Strength (psi)**
Masonry	1,463	-	1,950
Reinforcing Steel	29,000	60,000	-

3.2 Pushover Analysis

The test setup in [3] was designed to simulate the application of uniformly distributed out-of-plane loads on the M-00 wall. The out-of-plane distributed loads were applied through a displacement-controlled actuator with 500kN capacity. The target displacement was set at 75 mm in node 16 as shown in Fig 2, in order to best capture the force-displacement relationship.

Fig 2 illustrates the implementation of the EFM in SAP2000. The base and the top of the wall are considered fixed and pinned, respectively. The CMU pier was simulated using two beam elements connected by a rotational link at the mid height. The MultiLinear Elastic Link used is shown in Fig 3. Fig 4 illustrates aa comparison between the force-displacement backbone curve of the M-00 wall in SAP2000 (designated as resistance function in the legend) to the experimental results obtained in [3].

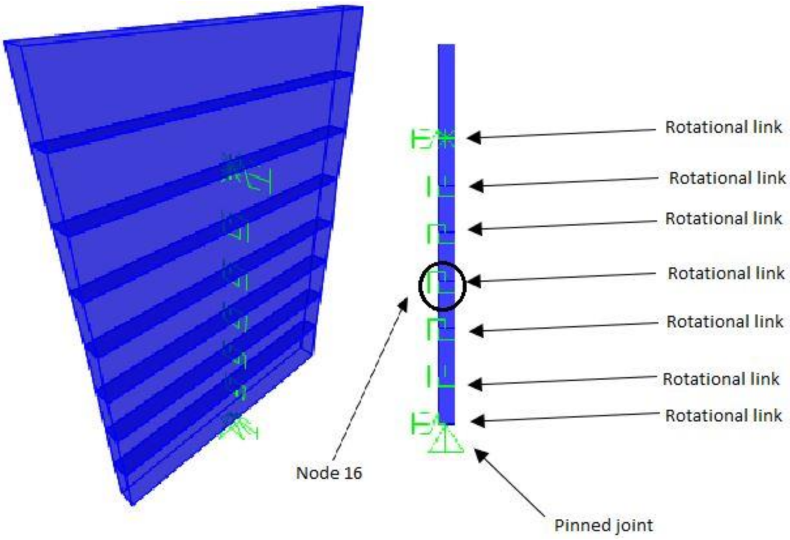
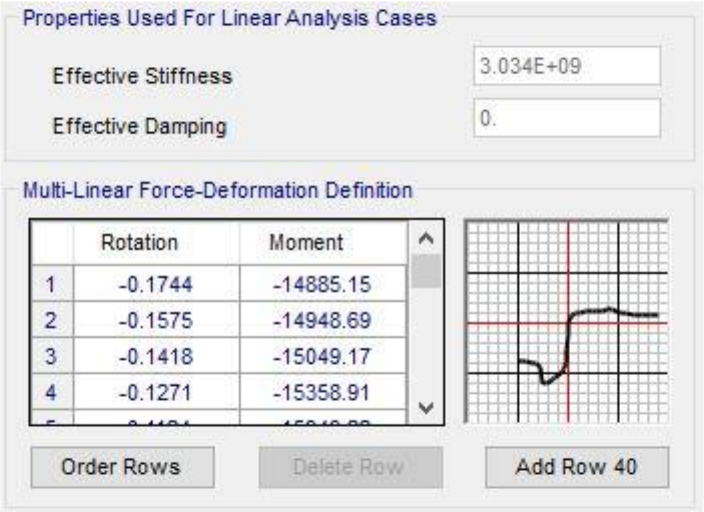


Fig. 2 – Equivalent Frame Method Model



Properties Used For Linear Analysis Cases

Effective Stiffness: 3.034E+09

Effective Damping: 0.

Multi-Linear Force-Deformation Definition

	Rotation	Moment
1	-0.1744	-14885.15
2	-0.1575	-14948.69
3	-0.1418	-15049.17
4	-0.1271	-15358.91

Fig. 3 – MultiLinear Elastic Link Properties Defined in SAP2000

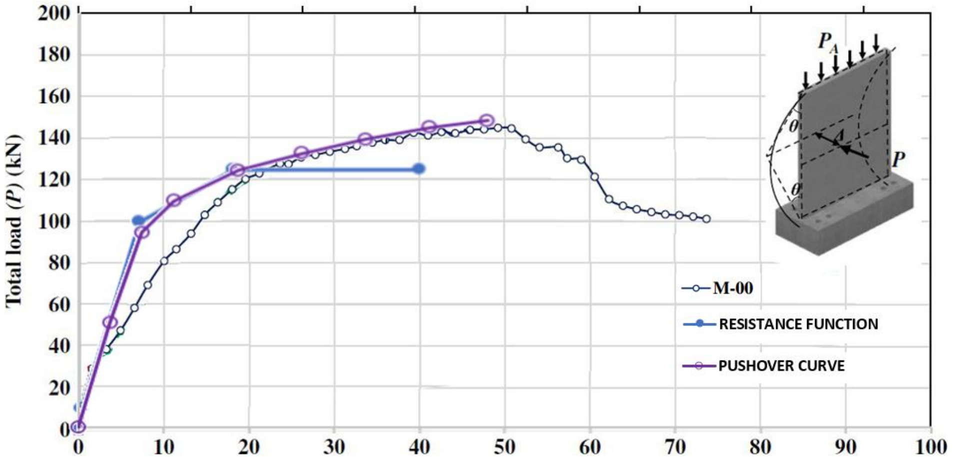


Fig. 4 – Force-Displacement Curve for the Wall [3]

4. Fragility Analysis

The assessment of component HCLPF fragilities is a crucial step in the Seismic Probabilistic Risk Assessment (S-PRA). The Conservative Deterministic Failure Margin (CDFM) method is widely used in engineering practice to estimate the HCLPF capacities of structures, systems and components (SSCs) in NPPs [27].

A HCLPF capacity corresponds to the earthquake level at which, with high confidence (≥ 95%), there is low probability (≤ 5%) that failure of SSCs required for safe shutdown of the plant will occur [28]. In this paper, the CDFM method was used since it is a code-based standard design analysis method that takes into account deterministic values of ground motion parameters of Review Level of Earthquake (PGA and response spectra ordinates) and design material properties (strength, damping value, ductility, etc.).

The out-of-plane fragility of the wall shown above was determined with the Review Level Ground Motion (RLGM) shown in Fig 5, with a PGA of 0.773g.

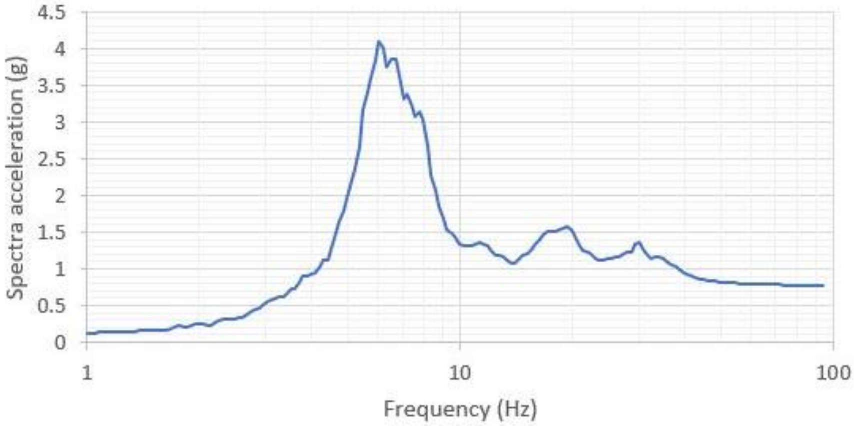


Fig. 5 – Review Level Ground Motion: 5% Damped Acceleration Response Spectra

The linear HCLPF capacity of the CMU wall was calculated from geometric and material properties presented above. The steel reinforcement area and the moment of inertia were incorporated in both the model and the analysis. For the calculation of the nonlinear HCLPF capacity of the CMU wall, data from the pushover analysis were used, such as the nominal moment capacity and the secant frequency. Using these data, a different spectral acceleration demand was obtained from the RLGM of Fig.5. The data used to calculate both capacities per the CDFM methodology are presented in Table 2.

Table 2 – Input Data for CDFM Method

Analysis	Moment capacity (kip-in/in)	Secant frequency (Hz)
Linear masonry	1.60	10.38
Resistance function	4.18	5.65
Pushover analysis	5.96	7.04

5. Results

When applying the CDFM method to calculate the HCLPF capacity, the 5% damped RLGM spectrum was used. Taking into account the data provided above, three HCLPF capacities were obtained. As can be seen in Table 3, the capacity calculated considering CMU wall nonlinearity is 2.16 times greater than the capacity calculated considering the CMU wall being fully elastic.

Table 3 – Results of CDFM Method

Analysis	Reference PGA (g)	Scale factor (g/g)	HCLPF (g)
Linear masonry	0.773	5.43	4.20
Resistance function	0.773	8.25	6.38
Pushover analysis	0.773	11.76	9.09

6. Conclusions

This paper presented the implementation of the CDFM method, using the results of a nonlinear analysis of a wall with out-of-plane seismic load. For comparison, a HCLPF value was also calculated from a linear analysis. The result of the method from an inelastic analysis is 116% higher than that obtained from the elastic analysis.

The incorporation of these non-linear results in the CDFM method depends on the non-linear properties of the wall and their implementation in the nonlinear model. It is demonstrated that higher and more realistic HCLPF capacities can be obtained when using the low computational cost EFM modeling approach.

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Any opinions, findings, conclusions, and recommendations presented in this report are those of the authors and do not necessarily reflect the views of the supports.

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