16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 4338 (Abstract ID)

Registration Code: S-V1464732943

USING RELIABILITY TO INCORPORATE MONOTONIC TEST DATA FOR A CODE COMPLIANT MASONRY LONG WALL DESIGN CRITERIA

G. C. Hart(1), C. C. Simsir(2), M. Rahmani(3)

(1) *Principal Emeritus, Thornton Tomasetti, ghart@thorntontomasetti.com*
(2) *Senior Associate, Thornton Tomasetti, csimsir@thorntontomasetti.com*
(3) *Project Engineer, Thornton Tomasetti, mrahmani@thorntontomasetti.com*

Abstract

This paper demonstrates using the equations of structural reliability theory and the quality rating approach in FEMA P695 how structural engineers when they wish to use long concrete masonry shear walls to resist earthquake loads can have confidence that their design meets the intent of the current building codes. In this paper, a proprietary masonry product used in shear walls is utilized to illustrate the incorporation of monotonic test data and demonstrate their code compliance with respect to in-plane loads. We define a long wall to be one that has a length to width ratio greater than one. This type of wall has also been called a low-aspect ratio wall in FEMA P695 and a squat wall by others. This paper demonstrates the development of an alternative design procedure for long walls that satisfies the acceptable probabilities of annual failure due to earthquakes as defined in ASCE 7-10. Various structural analysis and reliability techniques to validate the alternative design criteria and verify the code compliance of the walls are presented in this paper as well.

Keywords: Masonry shear walls; Shear-dominated walls; Structural reliability

1. Introduction

Traditionally, structural analysis, structural reliability theory, and the results of laboratory testing of structural components combine to develop a building design criteria for earthquake loads. With the worldwide creation of excellent structural component testing laboratories, with the development of even more sophisticated structural analysis models, and with faster computer speeds, we can utilize structural reliability theory perhaps even beyond its founders imagined in the 1950s. As required in the approach presented in this paper we now can use the peer review process on projects with experts around the world. As required in the approach presented in this paper the use of structural observation beyond minimal construction inspection can be rewarded because of the associated reduction in uncertainty in construction following building department approved structural plans. The presented criteria using limit states for shear and flexure provided transparency and also the direct incorporation of professional education and judgment from the structural engineer of record, the Peer Review members, and building officials.

What is important for us as structural engineers is to utilize all existing test data in developing our model even when the tests were conducted with a test plan we would not develop today. This valuable prior test data could be from monotonic tests, cyclic tests with a cyclic test protocol that we would not use today, instrumentation that did not measure all desired concrete or steel strains and deformations. Because test protocols change with time and from different testing laboratories and nations, we must use structural reliability theory to enable us to utilize the test data from others and develop design criteria. Just like when we go into the field as part of an earthquake reconnaissance team, we can learn a great deal from these tests just looking at the type of failure, e.g. diagonal shear cracks in a shear wall, vertical reinforcing bar post-earthquake location, concrete crushing at the end of a wall taller than it is long.

This paper utilizes a proprietary masonry product used in shear walls to illustrate the incorporation of monotonic test data and demonstrate their code compliance with respect to in-plane shear. We define a long wall to be one that has a length to width ratio greater than one. This type of wall has also been called a low-aspect ratio wall in FEMA P695 and a squat wall by others. Our preference for using long walls is that it describes that they are longer than taller and easier to communicate with non-structural engineers. As shown in the paper, we must have a Very Certain level of confidence that a long wall is shear-dominated, with a capacity for the flexure limit state that ensures it is the shear limit state that controls the performance of the wall. This basic objective is addressed in current codes with prescriptive provisions and committee member professional experience and judgment. This paper demonstrates the development of an alternative design procedure for long walls that satisfies the acceptable probabilities of annual failure due to earthquakes as defined in ASCE 7-10 [1]. This paper presents various structural analysis and reliability techniques to validate the alternative design criteria and verify the code compliance of the walls.

2. Long Walls

Reinforced masonry shear walls in seismically active regions typically fall into one of two limit state design categories: 1) Flexure-dominated (tall) shear walls: The mode of failure of the walls is flexure and the limit state is controlled by the maximum compressive strain in the masonry unit, 2) Shear-dominated (long) shear walls: The mode of failure of the walls is shear and the limit state is controlled by the shear stress capacity of the wall. This paper addresses the second category of walls and names them long walls. NIST report GCR 14-917-31 [2] provides further discussion of both tall and long shear walls. Fig.1 shows a typical configuration of a tall wall versus a long wall in elevation.

Building code requirements for reinforced masonry shear walls typically strive to achieve a design controlled by flexure. However, due to the wide variety of masonry wall types and configurations and the lack of control of the structural designer over these configurations in many cases, following the prescriptive requirements alone does not necessarily ensure ductile, flexure-dominated behavior [2]. While tall walls like the one shown in Fig.1a are most likely to display the flexure-dominated behavior that meets the intent of the building code, long walls like the one shown in Fig.1b with a wall length greater than its height are common, and they are often much stronger than required.

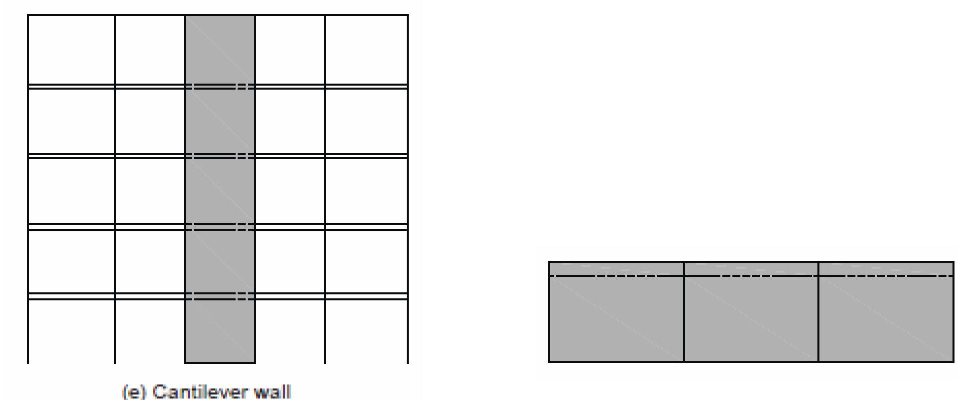

Fig. 1 – Typical configurations of a) tall versus b) long shear wall (NIST report GCR 14-917-31 [2])

Long walls with a wall length greater than its height, a shear-span-to-depth ratio less than one, or a high axial load are generally brittle, with failure characterized by diagonal shear cracks. This is often the case for low-rise masonry buildings, which represent the typical masonry construction in most places. Long walls have a greater flexural capacity than shear capacity.

Fig.2 shows a sample of cyclic load deflection test data for reinforced masonry shear walls failing in shear. The test results indicate the following: 1) a brittle failure mode after peak capacity; 2) a lack of numerous, fat hysteretic loops; 3) and a lack of cyclic deterioration prior to peak capacity. Prior to reaching peak capacity, the cyclic envelope curve essentially follows the monotonic loading curve with little, if any, degradation in strength or stiffness.

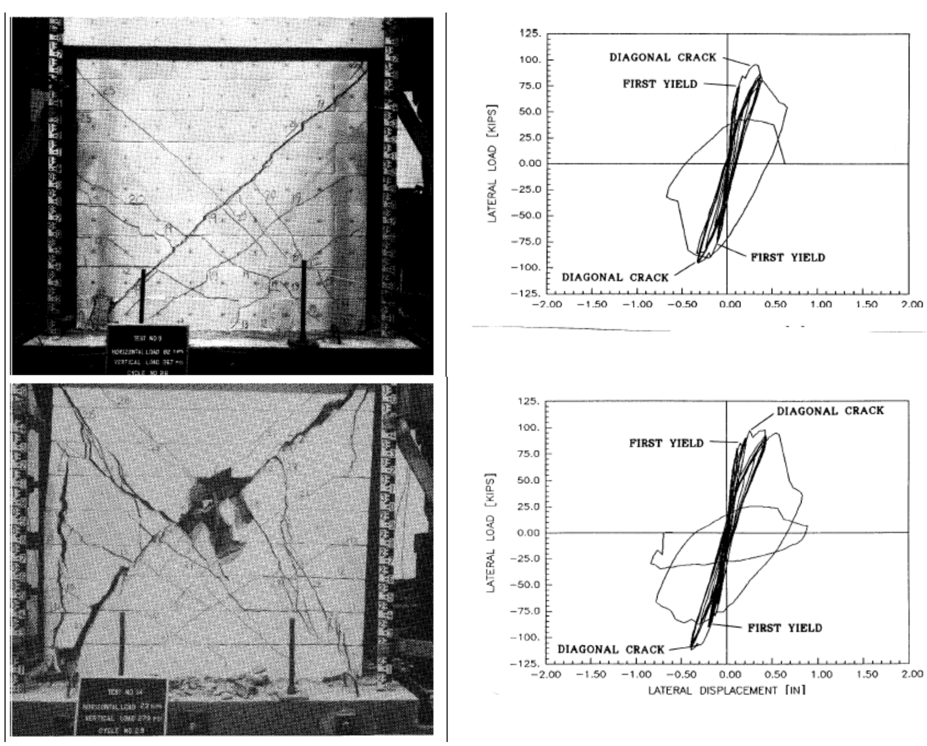

Fig. 2 – Cyclic tests on reinforced masonry walls failing in shear (FEMA 307 [3] and Shing et al. [4])

3. Proprietary Masonry Product

The proprietary masonry product used as an example for the purpose of this paper is called Korfil Hi-R and Hi-R-H. The proprietary Korfil insulation inserts are designed for use with the proprietary Hi-R and Hi-R-H hollow concrete masonry units (CMU). The insulation inserts are inserted into part of the cells of CMUs and are in direct contact with one face shell of CMUs, for non-structural purpose, such as for thermal resistance. The CMUs comply with ASTM C90, but have slightly different face shell and web thicknesses than standard CMUs, and their notched webs accommodate the insulation inserts and the horizontal reinforcement inside the CMU cells. Fig.3 shows nominal 12-inch (300 mm) wide Korfil Hi-R and Hi-R-H CMUs.

Fig.4 and Fig.5 compare with standard CMU the grouted cross-sectional areas and section moduli, respectively, of nominal 10-inch (250 mm) and 12-inch (300 mm) wide Hi-R and H-R-H. The proprietary CMUs have slightly smaller geometric properties than the standard CMU when the wall is grouted. Unlike a standard CMU which is symmetric about its centroidal longitudinal axis, the proprietary CMUs have their centroidal axis shifted towards the grouted side and away from the insulation side. Therefore, the section modulus is smaller for the insulation side than the grouted side of the proprietary CMUs (Fig.5).

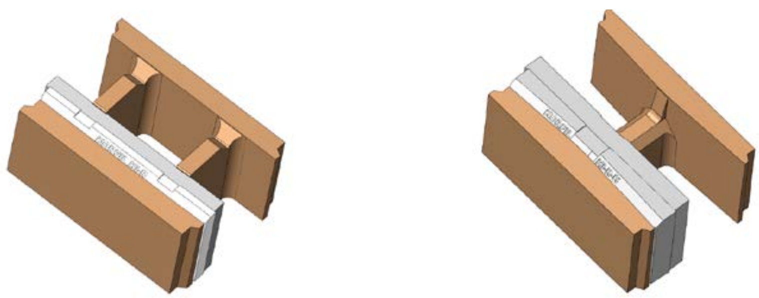

Fig. 3 – Nominal 12-inch (300 mm) wide a) Hi-R, and b) Hi-R-H concrete masonry units with inserts

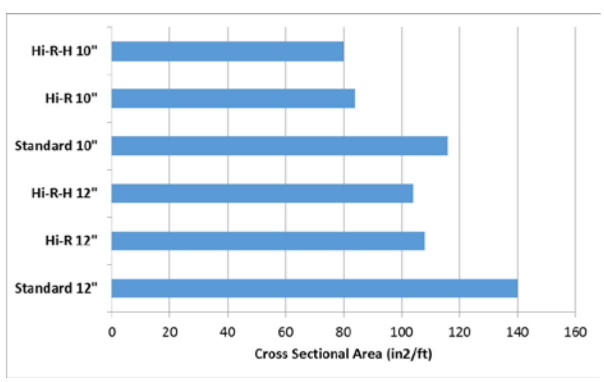

Fig. 4 – Comparison of CMU cross-sectional areas

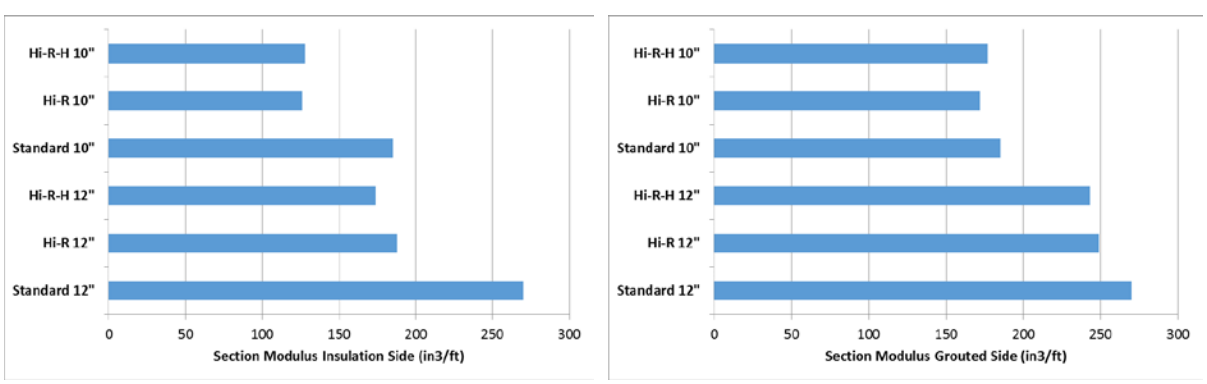

Fig. 5 – Comparison of CMU section moduli: a) insulation side, and b) grouted side

Walls constructed with this masonry product have been tested in various ways including axial compressive tests, flexural tests, in-plane shear tests, prism tests, etc. with varying amounts of grouting and horizontal and vertical reinforcement, as presented in the NCMA report (1986) [5]. However, only the wall tests for in-plane shear for solid grouted specimens with #4 bars at 24 inches on center (#13 at 610 mm) are shown in Fig.6 for brevity. These specimens were subjected to monotonic loading tests (Fig.6).

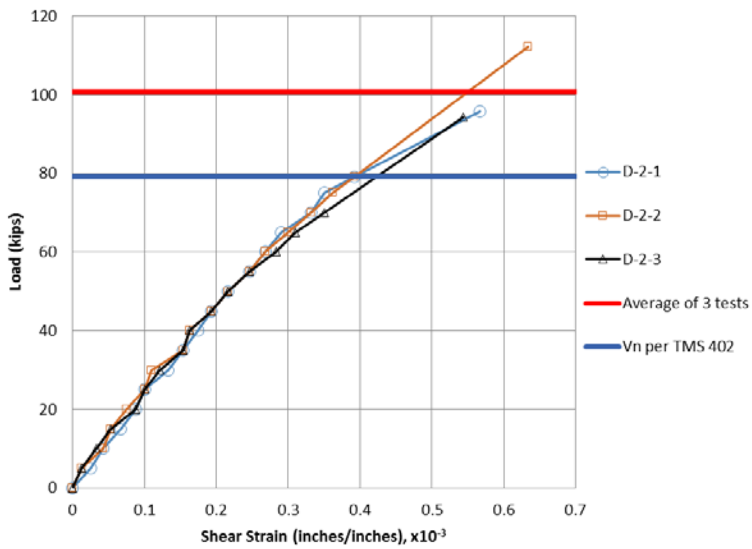

Fig. 6 – Load versus shear strain from wall shear tests (#4@24” o.c., insulation inserts, solid grouted)

4. Design Procedure Development

Acceptable alternative criteria were defined for developing a design procedure that satisfies the acceptable probabilities of annual failure due to earthquakes as defined in ASCE 7-10 for special reinforced masonry walls. The detailing and design provisions met the building code (IBC 2015 [6] and TMS 402-13 [7]) requirements, except as noted. These alternative design criteria were then validated as described in Section 5 of this paper.

A few iterations were made for the proposed design procedure during review of the masonry product under consideration and the current building codes and standards to see what modifications, if any, need to be made. A draft design procedure was developed, followed by modifications, to meet the required level of safety for the limit state. Presented here is the design procedure after a few iterations and prior to the verification of the design procedure. The proposed design provisions are presented here to illustrate that with a narrowing of the focus of a design procedure we can have less uncertainty and gain rewards in material reduction and typically cost reduction.

The proposed design brings several limitations beyond the requirements in the code in terms of the maximum steel bar size, solid grouting, and wall configurations, the details of which are presented in Hart and Simsir [8]. We note that with these limitations our confidence in performance significantly increases and this reduction in uncertainty is rewarded in the quality ratings for the design requirement as discussed in Section 5 of this paper.

Other important limitations are for the wall length to be greater than its height and the wall shear-span-to-depth ratio $M_u/(V_u d_v)$, as defined in TMS 402 [7], to be less than one. These requirements give more confidence in the performance of the wall as a shear dominated wall.

The proposed design calls for the use of R=5 and strength design load factors to calculate shear demand V_1, but then the design shear demand, V_u, is obtained by multiplying V_1 by 2.5. The building code for shear-dominated walls rather than setting a different and lower R value does in effect the same thing by taking the forces corresponding to the R=5 and multiplying them by 2.5. This in effect corresponds to an R value of 2. This is an indirect way of not counting on ductility in a long wall. Therefore, TMS 402 states that the nominal shear strength need not exceed 2.5 times the shear demand obtained from structural analysis.

The proposed design calculates the shear capacity of wall (V_c) per building code to include the contributions to capacity from nominal masonry and steel strengths. A capacity reduction value of ϕ for shear of 0.70 is used as determined per the procedure shown in Section 4.1, in lieu of 0.8 in the building code.

A design check is performed such that the ratio of shear capacity (V_c) to design shear demand (V_u) must exceed one. This checks the shear design to confirm that the wall has sufficient capacity to resist seismic demand. The shear-dominated wall performance is verified, one more time, as follows: The shear force corresponding to the development of 1.25 times the nominal flexural capacity of the wall ($1.25M_nV_1/M_1$) shall exceed the design shear capacity of the wall ϕV_c, where, V_1 is the base shear force demand based on R=5, and M_1 is the base overturning moment demand based on R=5. This verifies that the wall has sufficient flexural capacity and it will not fail in a flexure limit state. This is contrary to building code (TMS 402) which attempts to keep the shear corresponding to flexural capacity smaller than shear capacity. The 1.25 factor is consistent with current code provisions of TMS 402 and represents a safety to ensure shear-dominated response.

Standard installation requirements of the building code are acknowledged. Special inspection is standard in most high seismic regions and is repeated here to ensure that it is required. Structural observation is essential to ensure superior construction and since most good structural engineers visit the site to ensure that the constructed building load path functions as intended, it represents a minimal cost impact. Peer review is an added level of quality insurance that help make the design better and constructible. The inclusion of these provisions clearly results in a higher quality rating for the plans and thus is a reward for their use with the FEMA P695 approaches as discussed in Section 5 of this paper.

4.1 Capacity reduction factor

A capacity reduction factor, ϕ, for shear based on the available test data and the expected response specific to the proprietary masonry product was developed. The value of the capacity reduction factor was established based on the structural reliability (safety index) analysis. The analysis considered the available test data (mean and coefficient of variation), the test results with respect to the design code based equation, the number of test specimens, and a larger value of safety index of 4 to account for the shear failure mode of long masonry walls (as compared to a smaller value of 3.5 or less typically used for the flexural limit state in building codes based on the theory of structural reliability).

Reinforced masonry long walls fail in diagonal tension. Other forms of failure such as sliding, toe crushing, or rocking (commonly observed either in unreinforced masonry walls or flexure-dominated reinforced walls) is not typically observed for reinforced masonry long walls. Test results are accepted when they verify the diagonal tension failure mode in testing – the following equations assume this behavior.

The maximum tested capacity for each test and the expected (mean) value of these maximum capacities ($\bar{C}_{TEST}$) were determined using the test data (Fig.6). The TMS 402 [7] building code equation for nominal shear capacity ($\bar{C}_{TMS,EQ}$) includes the contribution of the masonry and the reinforcing steel. The ratio of the test capacity, $\bar{C}_{TEST}$, to this code equation with nominal material values is as follows:

$$\alpha_1 = \left(\bar{C}_{TEST} / \bar{C}_{TMS,EQ}\right) \geq 1.0 \tag{1}$$

The design equation for capacity using structural reliability theory is:

$$\text{Design Capacity} = \hat{\phi}\bar{C}_{TEST} \tag{2}$$

$$\hat{\phi} = \exp\left[-0.75\beta\rho_C\right] \tag{3}$$

β = Structural reliability (safety) index

ρ_C = Coefficient of variation of capacity

Substituting Eq. (1) into Eq. (2), we obtain the following:

$$\text{Design Capacity} = \left(\hat{\phi}\alpha_1\right)\bar{C}_{TMS,EQ} \tag{4}$$

Now define a capacity reduction factor as follows:

$$\phi = \hat{\phi}\alpha_1 \leq 0.8 \tag{5}$$

The value of 0.8 is the limit set per TMS 402 [7] for shear. It follows that

$$\text{Design Capacity} = \phi\bar{C}_{TMS,EQ} \tag{6}$$

A value of 0.70 for ϕ as noted earlier was used for the shear walls in lieu of ϕ = 0.8 for shear in the building code. Fig.7 shows a plot of ϕ versus α_1 and ρ_C for β = 4 for reference. We note that other acceptable methods exist to calculate values for reliability indexes, capacity reduction factor, or probability of failure for selected limit states.

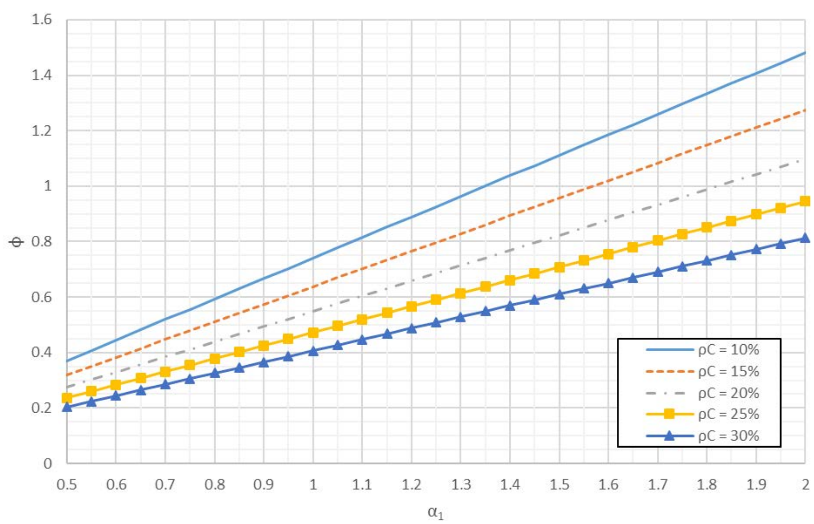

Fig. 7 – Variation of capacity reduction factor Ø with α_1 and ρ_c for $\beta = 4$

5. Verification of Design Procedure

This section describes the validation of the alternative design criteria developed in the preceding section in this paper by using the methodology and acceptance criteria of FEMA P695 [9] and performing calculations for the adjusted collapse margin ratio (ACMR) as defined in FEMA P695. An overview and evaluation of the FEMA P695 methodology is also presented in the NIST GCR 10-917-8 report [10].

Based on this procedure, the performance of a structural system is deemed acceptable if the probability of collapse due to maximum considered earthquake (MCE) ground motions is limited to an acceptably low value. Structural systems are required to meet a 10% collapse probability limit, on average, for typical, risk category II structures. Recognizing that some individual systems could have collapse probabilities that exceed this value, a limit of twice that value, or 20%, is used as the criterion for evaluating the acceptability of potential "outliers" within a group. High occupancy structures (risk category III) and essential facilities (risk category IV) require the more stringent limits of collapse probability of 6% and 3%, respectively. Table 1 reproduced from ASCE 7-10 [1] lists the acceptable probabilities of failure for earthquake loads for various risk categories of structures.

Table 1 – Anticipated reliability (maximum probability of failure) for earthquake (ASCE 7-10 [1])

Risk Category I and II	
Total or partial structural collapse	10% conditioned on the occurrence of Maximum Considered Earthquake shaking
Failure that could result in endangerment of individual lives	25% conditioned on the occurrence of Maximum Considered effects
Risk Category III	
Total or partial structural collapse	6% conditioned on the occurrence of Maximum Considered Earthquake shaking
Failure that could result in endangerment of individual lives	15% conditioned on the occurrence of Maximum Considered Earthquake shaking
Risk Category IV	
Total or partial structural collapse	3% conditioned on the occurrence of Maximum Considered Earthquake shaking
Failure that could result in endangerment of individual lives	10% conditioned on the occurrence of Maximum Considered Earthquake shaking

Two samples of long walls were selected from a 31-ft (9.4 m) tall, one-story warehouse located in California: Wall 1 has a length-to-height aspect ratio of approximately five and a shear-span-to-depth ratio (M_1/V_1d) of 0.12, while Wall 2 has a length-to-height aspect ratio of approximately one and a shear-span-to-depth ratio (M_1/V_1d) of 0.84. The two long walls were designed for in-plane shear using 12-inch (300 mm) wide Hi-R blocks.

Fig.8 shows the structural analysis models for the two walls developed using the finite element computer program FEM/I [11]. Fig.9 shows the load versus deformation (capacity) curves for the two reinforced masonry walls obtained from a nonlinear pushover analysis using the FEM/I software [11]. As expected, the longer wall (Wall 1) is more brittle with a more rapid decline in capacity.

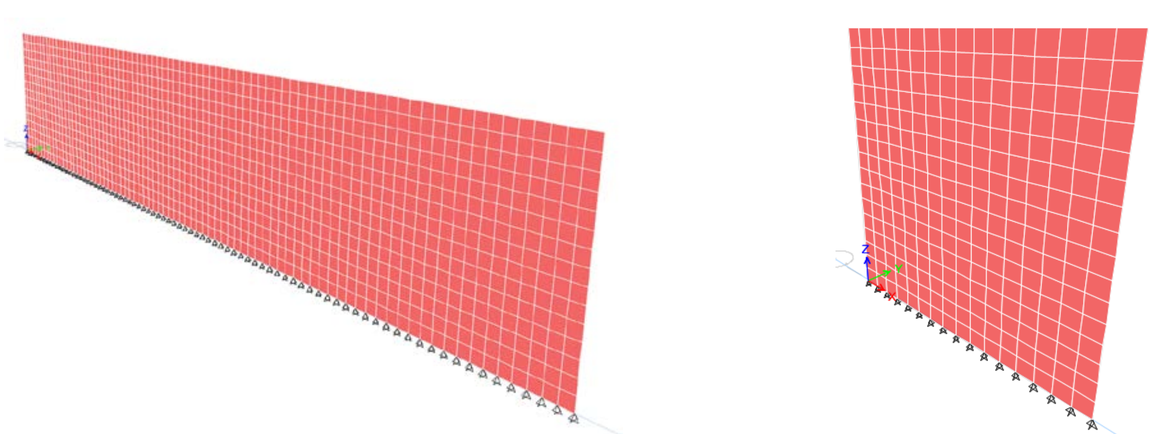

Fig. 8 – Finite element mesh for a) Wall 1 with aspect ratio of five; b) Wall 2 with aspect ratio of one

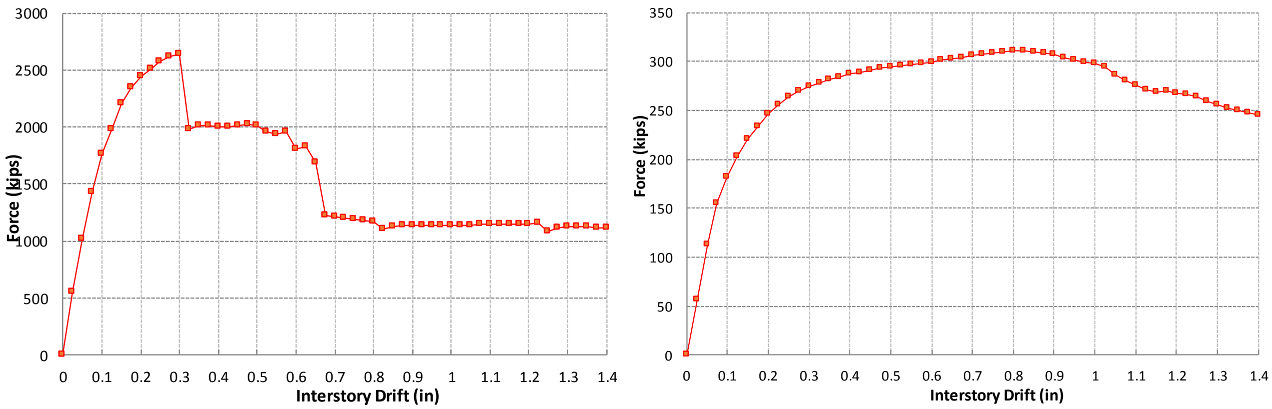

Fig. 9 – Capacity curve for a) Wall 1 with aspect ratio of five; b) Wall 2 with aspect ratio of one

The capacity curves for the two walls were then used as input for nonlinear response history analyses with increasing intensity of ground motions using the computer program IIIDAP [12]. Structural analysis models of the two walls with deteriorating hysteretic properties were developed in IIIDAP which was then used to perform an incremental dynamic analysis (IDA) on each wall. The IDA considered 40 ground motions (20 pairs in two orthogonal directions) provided by FEMA P695 [9] for the maximum considered earthquake (MCE) collapse limit state analysis. A sample of the IDA results are shown in Fig.10 and Fig.11. The computed wall peak displacements were plotted versus the input acceleration scale factors which form the IDA curves. Fig.11 shows the IDA curves for Wall 2 and also includes the mean (μ), the 16th percentile (μ - σ) and the 84th percentile (μ + σ) curves.

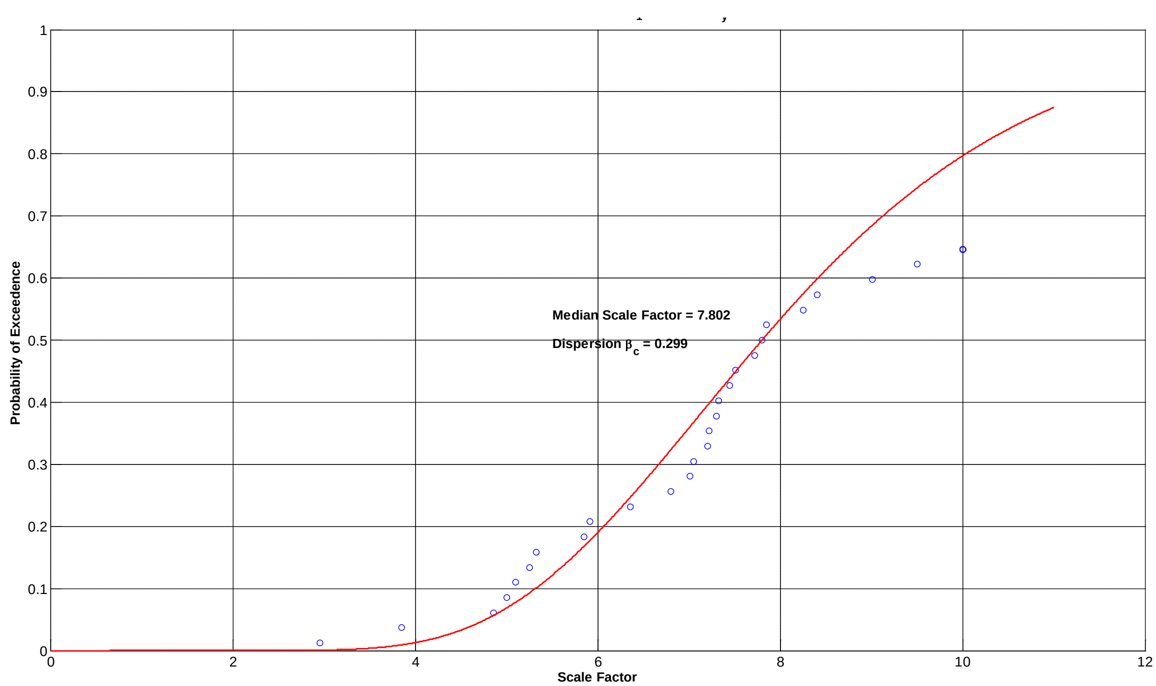

Fig. 10 – Wall 2 (aspect ratio of one) collapse fragility curve

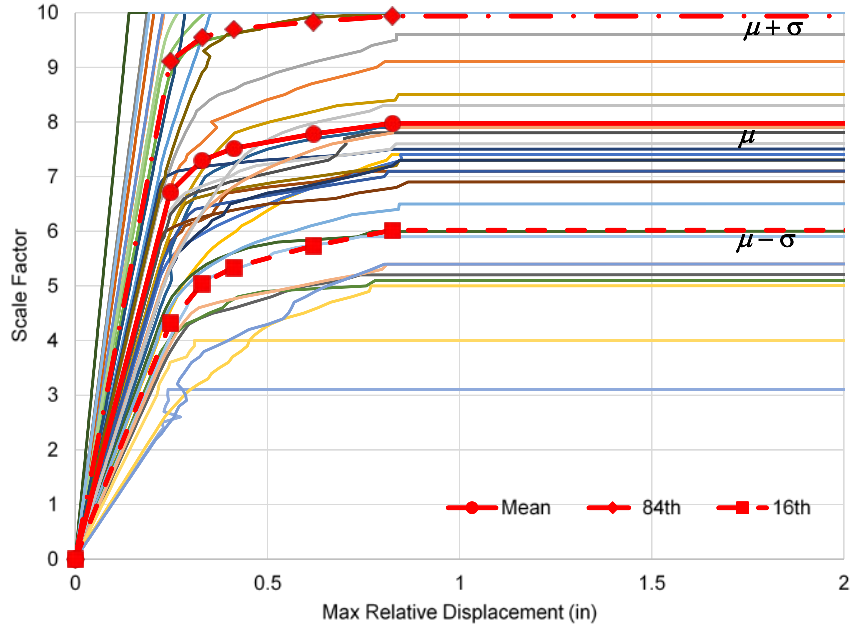

Fig. 11 – Wall 2 (aspect ratio of one) incremental dynamic analysis results

Fig.12 shows the normal distribution of ground motion acceleration scale factors associated with collapse level displacement (also called 100% target displacement herein) obtained from the IDA. Uncertainty in the calculated scale factors are shown using the 16th percentile and the 84th percentile lines on the curve (Fig.12). In addition to the collapse level (100% target) displacement, acceleration scale factors were obtained for 75% and 50% target displacements. Table 2 presents the median, mean, standard deviation, and coefficient of variation of the acceleration scale factors calculated for the IDA ground motions and the three target displacement levels. The results reveal that the uncertainty in the calculated acceleration scale factors increases as the target displacement decreases. This is mainly due to the limited displacement ductility and the rapid deterioration and significant loss of capacity of the long wall as it steps into the inelastic range.

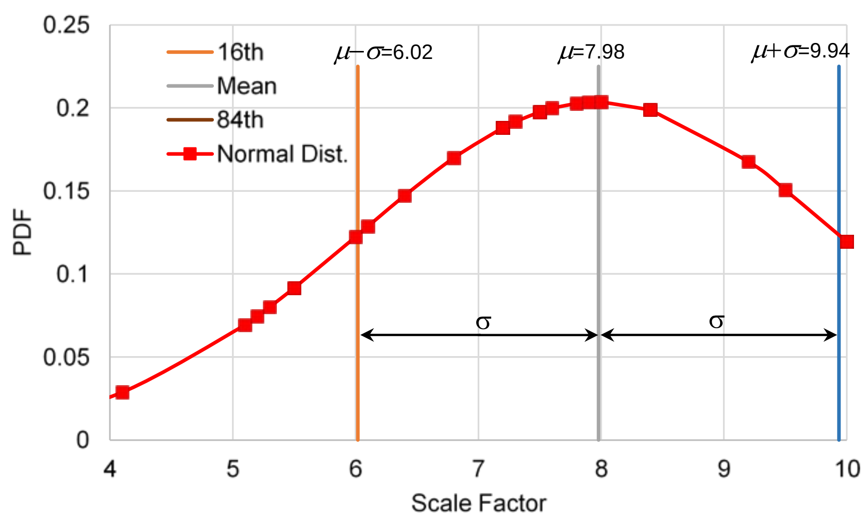

Fig. 12 – Normal distribution of collapse level ground motion acceleration scale factors

Table 2 – Statistical variation of acceleration scale factors of 40 ground motions calculated using Incremental Dynamic Analysis (IDA) for three target displacement levels

	100% Target Displacement	75% Target Displacement	50% Target Displacement
Median	7.90	7.50	7.20
Mean (μ)	7.98	7.78	7.52
STD (σ)	1.96	2.05	2.18
COV	24.6%	26.3%	29.0%

The median collapse intensities obtained from the IDA analysis were used to calculate the collapse margin ratio (CMR) per FEMA P695 [9]. CMR is the primary parameter used to characterize the collapse safety, taken as the ratio between the median collapse intensity and the MCE ground motion intensity. For a very long wall (Wall 1) and a wall just as long as it is tall (Wall 2) the resulting CMR values are 5.0 and 3.9, respectively, for a typical site class D in Los Angeles, California. The CMR values were adjusted for spectral shape by multiplying them by the spectral shape factor (SSF) per FEMA P695 based on fundamental period and ductility. The resulting adjusted collapse margin ratio (ACMR) for Wall 1 and Wall 2 is 5.65 and 4.76, respectively.

One way to incorporate the uncertainty and its significant impact on the final design is to consider the collapse total uncertainty (β_{TOT}) per FEMA P695 as a function of the uncertainties associated with the ground motion data, design requirements, test data, and structural models:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{7}$$

The uncertainties and their range of values are provided in FEMA P695 as follows:

β_{TOT} = total collapse uncertainty (0.275 - 0.950)

β_{RTR} = record-to-record collapse uncertainty due to variability in response to ground motions (0.20 - 0.40)

β_{DR} = design requirements-related collapse uncertainty associated with confidence in basis of design requirements (0.10 – 0.50)

β_{TD} = test data-related collapse uncertainty associated with confidence in test data (0.10 – 0.50)

β_{MDL} = modeling-related collapse uncertainty associated with confidence in accuracy and robustness of analytical models (0.10 – 0.50).

Based on the confidence we have on the design requirements, test data, and structural modeling discussed in the preceding sections, the following uncertainties were assigned for the two walls described in this paper:

β_{RTR} = 0.30 based on period-based ductility per FEMA P695

β_{DR} = 0.10 (Superior confidence)

β_{TD} = 0.20 (Good confidence)

β_{MDL} = 0.15 (Superior-to-Good confidence)

Based on the above uncertainties, the following technology transfer reward has been realized: 30% versus 40% for β_{RTR}; 10% versus 50% for β_{DR}; 20% versus 50% for β_{TD}; 15% versus 50% for β_{MDL}. Thus, a total collapse uncertainty of 0.40 was computed.

The acceptable values of ACMR for different collapse probabilities (Table 1) are specified in FEMA P695 based on total collapse uncertainty (β_{TOT}). These are as follows for β_{TOT} of 0.40:

Acceptable ACMR for 03% collapse probability = 2.0 (extrapolated)
Acceptable ACMR for 05% collapse probability = 1.93
Acceptable ACMR for 10% collapse probability = 1.67
Acceptable ACMR for 15% collapse probability = 1.51
Acceptable ACMR for 20% collapse probability = 1.40
Acceptable ACMR for 25% collapse probability = 1.31

The ACMR values calculated for the two walls were larger than these acceptable ACMRs in FEMA P695. Hence, the two walls met the maximum probability of collapse criteria of ASCE 7-10 and therefore their design procedure was acceptable.

6. Conclusions

This paper utilized a proprietary masonry product used in shear walls to illustrate the incorporation of monotonic test data and demonstrate their code compliance with respect to in-plane loads. This paper presented the development of an alternative design procedure for long walls that satisfied the acceptable probabilities of annual failure due to earthquakes as defined in ASCE 7-10. Median collapse intensities obtained from an incremental dynamic analysis were used to calculate the adjusted collapse margin ratios, which showed that the acceptable probabilities of annual failure were satisfied by a large margin indicating that the capacity reduction factor for long walls can be increased further in the alternative design procedure. This is dependent on the reduction of the total collapse uncertainty, which is based on the engineer's confidence on design requirements, test data, and structural modeling.

7. Acknowledgements

The authors would like to thank Concrete Block Insulating Systems (CBIS) for their support, and Mr. Daren Reyes and Mrs. Kendra Burnham for their significant contributions in the preparation of this paper.

8. References

[1] ASCE/SEI 7-10 (2010): *Minimum Design Loads for Buildings and Other Structures*. American Society of Civil Engnrs.

[2] NIST GCR 14-917-31 (2014): *NEHRP Seismic Design Technical Brief No. 9 – Seismic Design of Special Reinforced Masonry Shear Walls*. National Institute of Standards and Technology.

[3] FEMA 307 (1999): *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings – Technical Resources*. Federal Emergency Management Agency.

[4] Shing PB, Noland JL, Spaeh HP, Klamerus EW, Schuller MP (1991): Response of Single Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads. *TCCMAR Report No. 3.1(a)-2*.

[5] NCMA (1986): *Research Investigation of the Structural Properties of Korfil Hi-R Concrete Masonry, A Report to Korfil, Inc.* National Concrete Masonry Association.

[6] IBC 2015: *2015 International Building Code*, International Code Council.

[7] TMS 402-13/ACI 530-13/ASCE 5-13: *Building Code Requirements for Masonry Structures and Commentary*, Masonry Standards Joint Committee.

[8] Hart GC, Simsir CC (2016): *Strength Design of Reinforced Masonry Shear-Dominated Walls with Hi-R and Hi-R-H in Seismic Design Categories C, D, E, F*. Thornton Tomasetti Report 15-35.

[9] FEMA P695 (2009): *Quantification of Building Seismic Performance Factors*. Federal Emergency Management Agncy

[10] NIST GCR 10-917-8 (2010): *Evaluation of the FEMA P695 Methodology for Quantification of Building Seismic Performance Factors*. National Institute of Standards and Technology.

[11] Ewing RD, El-Mustapha AM, Kariotis JC (1987, Reformatted 2000): *FEM/I – A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components*.

[12] Lignos DG (2014): *Interactive Interface for Incremental Dynamic Analysis, IIIDAP: Theory and Example Applications Manual, Version 1.2*. Department of Civil Engineering, McGill University, Montreal, Quebec.