Strength Design of Reinforced Masonry Shear-Dominated Walls with Hi-R and Hi-R-H in Seismic Design Categories C,D,E,F

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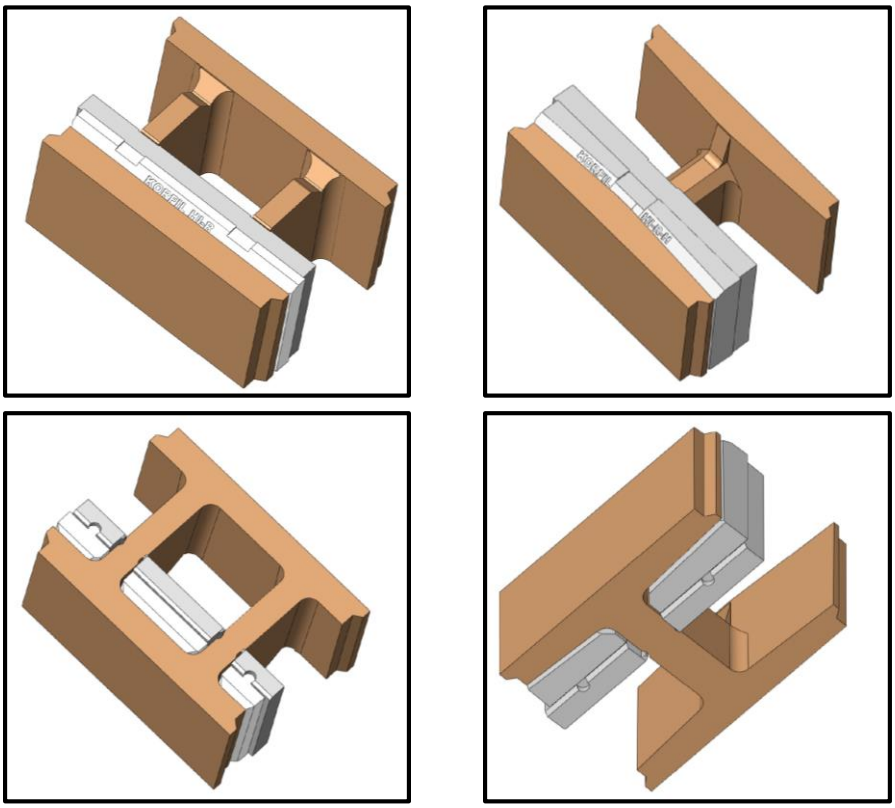

For

Concrete Block Insulating Systems, Inc.
P.O. Box 1000
Freight House Road, West Brookfield, MA 01585-1000

by

Gary C. Hart, PhD, PE
Principal Emeritus
Professor Emeritus, University of California, Los Angeles

C. Can Simsir, PhD, PE
Senior Associate

Thornton Tomasetti
4551 Glencoe Avenue, Suite 350
Marina del Rey, CA 90292
Phone 1.310.315.8430
Fax 1.310.315.8431
www.ThorntonTomasetti.com

FOREWORD

It has been my privilege to serve the masonry industry for over 40 years. This started in the early 1980s with the development of the first design provisions for the strength design for concrete masonry, funded in part by the Concrete Masonry Association of California and Nevada. Appendix N of this report quotes from a paper describing this process and the strength design provisions. This service was followed by the honor of being elected to the President of The Masonry Society.

I have now been given the honor to working with Dr. Can Simsir, a University of Illinois Ph.D. who studied under another past President of the Masonry Society Dan Abrams and has worked on my staff for over a decade. This report describes Dr. Simsir's and my best efforts to use our education and training to present a scientifically based design criteria for a very independent class of concrete masonry walls which are walls where the design limit state is shear failure. While many engineers call shear-dominated walls "squat walls", we prefer to call them Long Walls.

We have been given the opportunity to develop this work and share it in this report by Mr. David Nickerson and Concrete Block Insulation Systems, Inc.

Gary C. Hart
Principal Emeritus, Thornton Tomasetti
and
Professor Emeritus, University of California, Los Angeles

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Supplemental Report: Reference Material for Shear-Dominated Walls Report

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1. PREFACE

The design of concrete masonry walls in earthquake country typically fall into one of two limit state design categories. One is the wall where the mode of failure is flexure and the limit state is controlled by the maximum compressive strain in the concrete masonry unit. The other is the wall where the length is long compared to the wall height. In this case, the limit state for the wall is a shear limit state controlled by the shear stress capacity of the wall. It is the design of the second category of wall that is addressed in this report. We shall call such a wall a Long Wall. An excellent discussion of both flexure and shear dominated walls is provided in the NIST (2014) Technical Brief by Gregory Kingsley, Benson Shing, and Thomas Gangel. Other references are given in Chapter 5.

Long (squat) walls constructed of concrete masonry and designed using strength design have performed very well in strong earthquakes even though the failure mode is a brittle one. The reason for such excellent performance is threefold. First, the code starts with an *R* value of 5 and in effect reduces the "Effective *R* Value" by 60% by multiplying the shear force demand by 2.5. This is an indirect way of not counting on ductility in a Long Wall when doing a design. Second, the code is based on the theory of structural reliability and this, in a very direct and transparent manner, addresses this limit state by assigning a Reliability (Safety) Index Value of 4, compared to 3.5 or less for the flexural limit state. Third, because of the performance of the wall under design load, the strain in the steel reinforcement is at first yield, the wall has not reached diagonal crack failure, and therefore the cyclic degradation is small, if at all.

The reader will note that the proposed design criteria not only limits the size of the units to a 10 inch minimum width to provide excellent grout coverage for the rebar, but also goes well beyond the code (Appendix C) in ensuring the quality of both design and construction because it requires the Peer Review of the Structural Design and Special Inspection of the Construction. Both of these quality control benefits are at the foundation of what is called Performance Based Design and also have a minimal impact on the project cost for squat walls and so the benefit / cost ratio is excellent.

We define an acceptable alternative criteria for developing a design procedure that satisfies the acceptable probabilities of annual failure due to earthquakes as defined in ASCE 7-10, Table 12.2-1 for Special Reinforced Masonry Walls. The detailing and design provisions shall meet TMS 402 except as noted. We have validated this alternative design criteria by performing the calculations for the Adjusted Collapse Margin Ratio using FEMA P-695, June 2009, "Qualifications of Building Seismic Performance Factors". The Strength Design Procedure in this report has been verified (Appendix G) to meet the Minimum Probability of Collapse Criteria in ASCE 7-10 Table C.1.3.1b.

This report also contains summaries of the author's resumes. Complete versions of their resumes are available upon request.

A companion report entitled "Supplemental Report: Reference Material for Shear-Dominated Walls Report" is provided for readers who wish to use our library of quotations to gain more information about Performance Based Design, Structural Reliability and Masonry.

2. THE DESIGN PROCEDURE

The design of concrete masonry structures, in which the lateral force resisting system is primarily comprised of Korfil Hi-R and Hi-R-H concrete masonry block long walls, shall follow the requirements of Sections 2104, 2105, 2106, and 2108 (strength design) of the International Building Code, except as modified by the following provisions:

1. Design Parameters
   a) Solid grouting: Walls shall be fully grouted.
   b) Hi-R and Hi-R-H concrete block size shall be 10" or 12" and laid in running bond.
   c) Reinforcing steel bar size shall be #8 maximum and vertical bars shall be doweled into footing.
   d) Masonry prism specified compressive strength ($f_m^{'}$) shall be 1500 psi minimum.
   e) Walls shall be planar (rectangular) with no skew.
   f) Wall length shall be greater than its height.
   g) Shear-span-to-depth ratio $M_u/(V_u d_v)$, as defined in TMS 402-13, shall be less than one.
   h) TMS 402-13 minimum horizontal and vertical steel reinforcement requirements shall apply.

2. Earthquake Shear Force Demand on Wall (V_u)
   a) Use R=5 and strength design load factors to calculate shear demand V_1.
   b) The design shear demand, V_u, is obtained by multiplying V_1 by 2.5.

3. Earthquake Shear Capacity of Wall (V_c)
   a) Calculate capacity from masonry using nominal masonry prism strength (V_m) from Eqn. 9-24 of TMS 402-13.
   b) Calculate capacity from steel using nominal steel yield strength (V_s) from Eqn. 9-25 of TMS 402-13.
   c) The nominal shear capacity (V_c) is obtained by $V_c = (V_m + V_s)$ from TMS 402-13 with the limitation that ($V_m + V_s$) cannot exceed 6√f'm
   d) Value of ϕ for shear shall be taken as 0.70.

4. Design Check

   The Ratio of Shear Capacity (V_c) to Design Shear Demand (V_u) must exceed one.

5. Verification of Shear-Dominated Wall Performance

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.

6. Quality Control

a) Specified compressive strength, f'_m, used in design shall be verified by testing as outlined in the building code. In addition, the compressive strain during testing shall be monitored for the following: Load versus deflection compressive tests on masonry prism (or concrete block) strength shall be performed on three test specimens prior to start of construction to verify that the minimum tested 28 day maximum compressive strength of each specimen exceeds the specified compressive strength and that the strain at maximum compressive strength exceeds 0.0015. The strain at failure (crushing) of the specimens shall exceed 0.0025.

b) Installation: Wall construction must comply with Section 2104 of the International Building Code.

c) Special Inspection: Wall construction must comply with Section 2105 and 1705 of the International Building Code.

d) Structural Observation: Construction shall have structural observation by a licensed civil or structural engineer and comply with Section 1704 of the International Building Code.

e) Peer Review: Design must be peer reviewed by a licensed civil or structural engineer.

3. THE DESIGN PROCEDURE COMMENTARY

3.1 Design Parameters

a) Solid grouting was shown over a decade ago on a study by Mr. Stuart Beavers of the Concrete Masonry Association of California and Nevada to have minimum if any impact on cost of construction. The elimination of thinking about which cells should be grouted in a partially grouted wall plus the repetitive nature of a solid grouted wall was identified in that study to increase the speed and quality of construction.

b) The limitation of the provisions to 10" and 12" walls is arbitrary and certainly the case could be made that the 10" minimum could be replaced by 8" walls. However, the extra grout and increased distance from reinforcing bar to insulation insert and faceshell is beneficial for both in-plane and out-of-plane performance of the wall. The requirement for running bond and not stack bond is a recommendation for all masonry in seismic zones.

c) The limitation in maximum bar size to be #8 will probably have no impact on the design developed and is consistent with the bar size in the tests described in Appendix F.

d) This limitation should have no impact on the design developed for a project.

e) It is always possible to develop a design using rectangular walls instead of T or L or I shaped walls. As with all shear walls constructed with masonry or concrete we always have more confidence in the performance of rectangular walls. The control joints between rectangular walls have been addressed for years and the detailing is available in the masonry literature.

f) This wall height to wall length ratio could be relaxed to be as low as walls with length half the height of the wall and such walls are often called flexure / shear walls. Extra care must be given to detailing at the base of the wall because under certain conditions the wall is a flexure dominated wall with a plastic hinge at the base of the wall. The limitation is here to make it easy for the structural engineer to develop his or her design in association with the architect and also when combined with the requirement in (g) gives extra insurance that the wall will perform as designed.

g) This provision has long been recognized as a design check to ensure that the wall will perform as a shear dominated wall.

h) This provision reinforces the idea that no matter what calculations might allow it is good to satisfy this long recognized as beneficial limitation in high seismic zones.

3.2 Earthquake Shear Force Demand on Wall

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.

3.3 Earthquake Shear Capacity of Wall

The equation to calculate the capacity of shear wall from the masonry and the steel is in the building code TMS 402-13, Equations 9-21 through 9-25.

The value of the capacity reduction factor was established based on the structural reliability analysis described in Appendix E:

The modified safety index procedure based on structural reliability analysis takes into consideration the available test data (mean and coefficient of variation), the test results with respect to the design code based equation, the cyclic versus static test data, the number of test specimens, and a larger value of safety index to account for the shear failure mode of long masonry walls (as compared to a smaller value typically used for flexure-dominated walls).

Reinforced masonry long walls will fail in diagonal tension. Other forms of failure such as sliding, toe crushing, or rocking (commonly observed in unreinforced masonry walls and flexure-dominated reinforced walls) will not be typically observed for reinforced masonry long walls where the vertical steel is doweled into the footing as required in the design – see 1 (c). Test results are accepted when they verify the diagonal tension failure mode in testing – the equations assume this behavior.

3.4 Design Check

Item 4 of the Design Procedure checks the shear design to confirm that the wall has sufficient capacity to resist seismic demand.

3.5 Verification of Shear-Dominated Wall Performance

Item 5 of the Design Procedure verifies that the wall has sufficient flexural capacity and it will not fail in a flexure limit state. The 1.25 factor is consistent with current code provisions of TMS 402-13 and represents a safety to ensure shear-dominated response.

3.6 Quality Control

a) The proposed material testing requires not only a verification of the maximum strength but also a verification of the material to have a strain capacity consistent with that used for design. The material can be either the masonry prism or the concrete block, as both are accepted test materials by the building code to ensure quality of masonry used in construction.

b) Standard installation requirements of the building code are acknowledged.

c) Special inspection is standard in most high seismic regions and is repeated here to ensure that it is required.

d) 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.

e) Peer review is an added level of quality insurance that help make the design better and constructible.

4. DEFINITIONS AND NOTATIONS

Collapse Margin Ratio (CMR): The primary parameter used to characterize the collapse safety of a system, taken as the ratio between the median collapse intensity and the Maximum Considered Earthquake (MCE) ground motion intensity.

Design Requirements-Related Uncertainty (β_{DR}): Collapse uncertainty associated with the quality of the design requirements of the system of interest.

First Diagonal Crack Strength: The shear force at the occurrence of greater than 50% change in tangent stiffness with respect to the initial stiffness as determined per test data.

Force-Controlled Action: An action that is <u>not allowed to exceed the nominal strength</u> of the element being evaluated. (ASCE 41-13, pg. 5) [Underline by Hart]

Low-Deformability Component: A component whose deformability is 1.5 or less. (ASCE 41-13, pg. 6)

Low-Rise Building: Enclosed or partially enclosed buildings that comply with the following conditions:
1. Mean roof height *h* less than or equal to 60 ft (18 m).
2. Mean roof height *h* does not exceed least horizontal dimension. (ASCE 7-10, pg. 241)

Maximum Considered Earthquake (MCE) Ground Motions: The most severe earthquake effects considered, as defined by Section 11.4 of ASCE/SEI 7-10.

Modeling Uncertainty (β_{MDL}): Collapse uncertainty associated with the quality of the analytical models.

Nominal Strength (as defined per ASCE 7-10, pg. 1): The capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Nominal Strength (as defined per ASCE 41-13, pg. 6): The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics; or (2) field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Non-Structural Inserts: Proprietary non-structural components, such as molded expanded polystyrene (EPS) blocks, that are inserted into part of the cells of CMUs and are in direct contact of one face shell of CMUs, for non-structural purpose, such as for thermal resistance. With the inclusion of non-structural inserts into cells of CMUs, grout is not directly bonded to one face shell that is in direct contact with the non-structural inserts. Non-structural inserts include, among others, Korfil insulation inserts produced by CBIS and used with Hi-R and Hi-R-H concrete block masonry.

Record-to-Record Uncertainty (β_{RTR}): Collapse uncertainty due to variability in response to different ground motions.

Shear-Dominated Wall: A wall whose response is dominated by diagonal shear (tension) cracks. The wall has a greater flexural capacity than shear capacity.

Test Data-Related Uncertainty (β_{TD}): Collapse uncertainty associated with the quality of the test data for the system of interest.

A, A_n or A_{nv}: Shear area of wall

A_v : Area of horizontal steel

B or β : Reliability (safety) index

C_{TEST} : Capacity of wall in shear as determined by testing

$C_{TMS,EQ}$: Capacity of wall in shear as determined by code equations

d or d_v : Depth of member in direction of shear (Length of shear wall)

E_m : Modulus of elasticity of masonry wall

E_v or G : Shear modulus of masonry wall

f'_m : Masonry specified compressive strength

f_y : Steel specified yield strength

h or H : Height of wall

L or ℓ : Length of wall

M_1 : Base overturning moment demand (based on R=5)

$M_u/(V_u d_v)$: Shear span-to-depth ratio

M_n : Nominal flexural (moment) capacity

M_u : Flexural (moment) demand

n : Number of test specimens

P_u : Axial (vertical) load demand (positive for compressive load)

R : Response modification coefficient

s : Spacing of reinforcing steel

t : Thickness of wall

V or ρ_C : Coefficient of variation of capacity

V_c or V_n : Nominal shear capacity

V_m or V_{nm}: Shear resistance provided by masonry

V_s or V_{ns}: Shear resistance provided by reinforcing steel

V_t : Coefficient of variation of maximum strength

V_u : Design shear force demand ($2.5V_1$)

V_1 : Base shear force demand (based on R=5)

α_1 : C_{TEST} / $C_{TMS,EQ}$

Ø : Strength reduction factor for shear

5. REFERENCES

AC15 (Approved 2010, editorially revised 2015), Acceptance Criteria for Concrete Floor, Roof, and Wall Systems and Concrete Masonry Wall Systems, International Code Council Evaluation Service.

ASCE/SEI 7-10 (2010), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers.

ASCE/SEI 41-13 (2013), Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers.

Brandow, G., Ekwueme, C. and Hart G., 2012 Design of Reinforced Masonry Structures, Concrete Masonry Association of California and Nevada.

Engineering Evaluation of Hi-R and Hi-R-H Masonry Wall Systems for Seismic Design Categories A and B per Allowable Stress Design (2014), Report No. 14-05, Weidlinger Associates, Inc.

FEM/I (1987, Reformatted 2000) A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components by R.D. Ewing, A.M. El-Mustapha, J.C. Kariotis.

FEMA 306 (1999) Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings – Basic Procedures Manual (ATC-43 Project), Federal Emergency Management Agency.

FEMA 307 (1999) Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings – Technical Resources, Federal Emergency Management Agency.

FEMA P440A (2009), Effects of Strength and Stiffness Degradation on Seismic Response, Federal Emergency Management Agency.

FEMA P695 (2009), Quantification of Building Seismic Performance Factors (ATC-63 Project), Federal Emergency Management Agency.

FEMA P-795 (2011), Quantification of Building Seismic Performance Factors: Component Equivalency Methodology (ATC-63-1 Project), Federal Emergency Management Agency.

Ibarra, L.F., Medina, R.A., Krawinkler, H. (2005), Hysteretic Models that Incorporate Strength and Stiffness Deterioration, Earthquake Engineering and Structural Dynamics, 34: 1489-1511.

IBC 2015, 2015 International Building Code, International Code Council.

ICC-ES Report ESR-3508 (2015), Korfil Hi-R and Korfil Hi-R-H Insulation Inserts for Proprietary Masonry Wall Systems, International Code Council Evaluation Service.

LATB-1 (2015), Performance Based Evaluation and Strengthening with Structural Reliability Foundation, Second Edition, Developed for Los Angeles Tall Buildings Structural Design Council by Gary C. Hart.

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

NIST GCR 10-917-8 (2010), Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors, National Institute of Standards and Technology.

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.

PEER/ATC-72-1 (2010), Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings, Pacific Earthquake Engineering Research Center / Applied Technology Council.

Research Investigation of the Structural Properties of Korfil Hi-R Concrete Masonry, A Report to Korfil, Inc. (1986), National Concrete Masonry Association.

Shing, P.B., Noland, J.L., Spaeh, H.P., Klamerus, E.W., and Schuller, M.P. (1991), Response of Single Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads, TCCMAR Report No. 3.1(a)-2.

TMS 402-13/ACI 530-13/ASCE 5-13 Building Code Requirements for Masonry Structures and Commentary

Yokel, F. Y. et al. (1970), Compressive Strength of Slender Concrete Masonry Walls, NBS Building Science Series 33.

APPENDIX A THE SHEAR AREA OF KORFIL HI-R AND HI-R-H WALLS

Figures A-1 and A-2 show Hi-R and Hi-R-H concrete blocks without grout and the direction of the shear force. The area highlighted in yellow in these figures is the grouted area of the masonry. The shear area then is the sum of the cross-sectional areas established by the block (face shells and webs) and the grouted cells. Tables A-1 and A-2 illustrate the shear areas by the Hi-R and Hi-R-H blocks, respectively, that provide the shear resistance provided by the masonry, V_{nm}, for shear area, A_{nv}, per TMS 402-13 Section 9.3.4.1.2.1. The requirements in this report are for fully grouted walls and therefore the grouted cell spacing must be taken as 8 inches in Tables A-1 and A-2 (as reproduced below from ESR-3508 and marked for convenience. Weidlinger report no. 14-05 dated June 25, 2014 provides further data on the properties of the Hi-R and Hi-R-H blocks with comparisons to regular blocks. This information is also reproduced below.

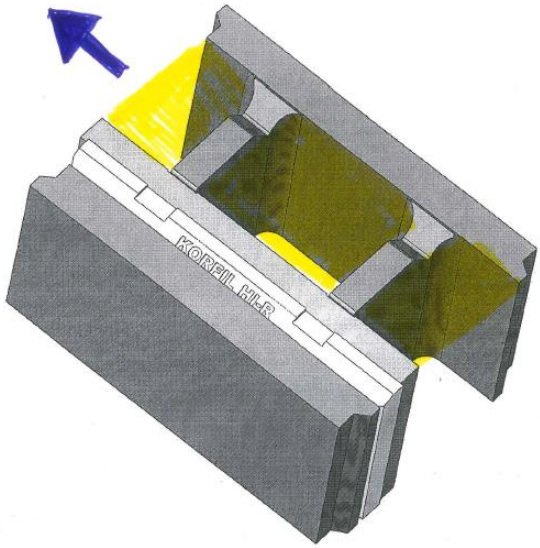


Figure A-1 Horizontal Shear Force (Korfil 12" Hi-R)

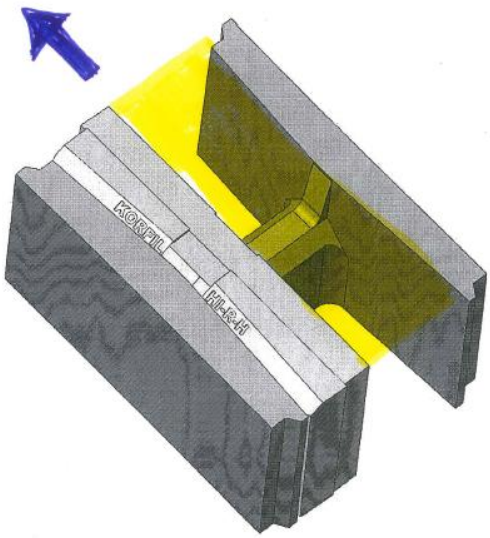


Figure A-2 Horizontal Shear Force (Korfil 12" Hi-R-H)

Tables A-1 and A-2 Sectional Properties of Hi-R and Hi-R-H (Reproduced from ESR-3508)

TABLE 1—SECTIONAL PROPERTIES OF KORFIL HI-R MASONRY UNITS WITH INSULATION INSERTS PROPERTIES PER FOOT OF WALL

NOMINAL BCLOK WIDTH (inches)	GROUTED CELL SPACING (inches)	NET CROSS SECTIONAL AREA[1] ($inches^2$)	NET CROSS SECTIONAL AREA - PERCENT SOLID	NET MOMENT OF INERTIA ($inches^4$)	SECTION MODULUS ($inches^3$)	
					Insulation Side	Grouted Side
10	8	84.0	72.7	700	126	172
	16	63.8	55.2	665	124	157
	24	54.5	47.2	647	123	148
	32	49.9	43.2	637	123	143
	40	47.1	40.8	630	123	140
	48	45.3	39.2	626	123	138
12	8	108.0	77.4	1245	188	249
	16	80.2	57.5	1165	181	225
	24	67.4	48.3	1126	179	212
	32	61.1	43.8	1105	178	205
	40	57.3	41.0	1092	177	200
	48	54.7	39.2	1084	177	197

[1]Section properties are based on net bedded area with cross webs adjacent to grouted cells having mortared bed joints.
SI Units: 1 in. = 25.4 mm; 1 in^2 = 645 mm^2; 1 in^3 = 16387 mm^3.

TABLE 2—SECTIONAL PROPERTIES OF KORFIL HI-R-H MASONRY UNITS WITH INSULATION INSERTS. PROPERTIES PER FOOT OF WALL

NOMINAL BLOCK WIDTH (inches)	GROUTED CELL SPACING (inches)	NET CROSS SECTIONAL AREA[1] ($inches^2$)	NET CROSS SECTIONAL AREA - PERCENT SOLID	NET MOMENT OF INERTIA ($inches^4$)	SECTION MODULUS ($inches^3$)	
					Insulation Side	Grouted Side
10	8	80.0	69.3	716	128	177
	16	62.3	54.0	681	126	160
	24	54.2	46.9	662	125	150
	32	50.2	43.4	652	125	145
	40	47.7	41.3	645	125	142
	48	46.1	39.9	641	125	139
12	8	104.0	74.6	1178	174	243
	16	76.2	54.7	1095	168	217
	24	63.4	45.4	1054	166	202
	32	57.1	40.9	1032	165	195
	40	53.3	38.1	1018	164	189
	48	50.7	36.3	1010	164	186

[1]Section properties are based on net bedded area with cross webs adjacent to grouted cells having mortared bed joints.
SI Units: 1 in. = 25.4 mm; 1 in^2 = 645 mm^2; 1 in^3 = 16387 mm^3.

COMPARISON OF HI-R-H AND HI-R MASONRY BLOCKS

Figures A-3 and A-4 show the general views and dimensions of the 12” wide Hi-R-H and Hi-R masonry blocks, respectively. The Hi-R-H block is an open ended unit and has one thick web connecting the face shells, while the Hi-R block has two webs. Figures A-3 and A-4 also illustrate in color the face shells (blue), the grouted cells (green), and the insulation inserts (pink) of the blocks.

Tables A-3 and A-4 list the geometric properties of the fully grouted, 12” and 10” wide Hi-R-H, Hi-R, and the standard block. The geometric properties include the cross-sectional area, the moment of inertia, and the section moduli per foot length of wall for out-of-plane bending (or about the centroidal axis running parallel to the length of the wall). The numbers in Tables A-3 and A-4 take into account the two face shells of the blocks, but do not consider the webs of the blocks which have little contribution to out-of-plane bending strength.

The comparison of the geometric properties of the masonry blocks with each other shows that the Hi-R-H block has slightly smaller geometric properties than the Hi-R block for grouted walls. Table 1 shows that the grouted Hi-R-H block has 3.7% smaller cross-sectional area than the grouted Hi-R block, which affects its compressive and shear stress properties. The grouted Hi-R-H has 2.4% to 7.4% smaller section moduli than the grouted Hi-R, which affect its flexural stress property.

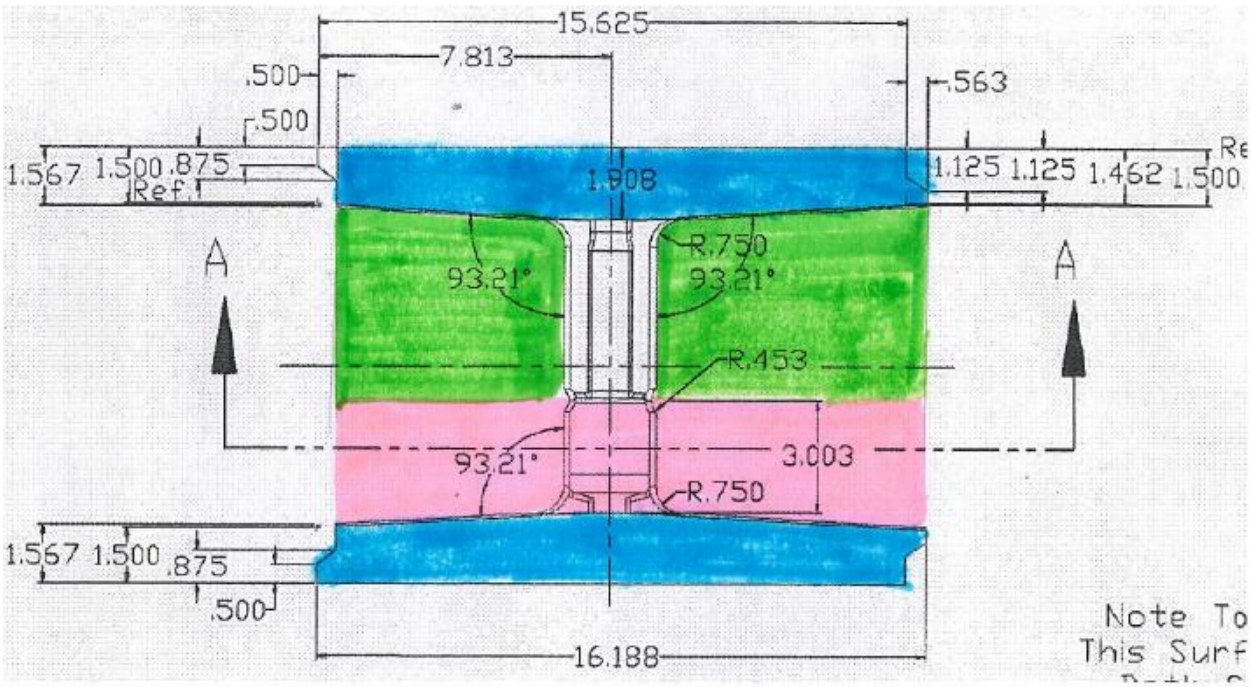
Both Hi-R-H and Hi-R blocks have smaller geometric properties than the standard block when the wall is solid grouted. The Hi-R-H and Hi-R blocks achieve comparable geometric properties with the standard block because they have slightly thicker face shells than a standard block, thus compensating for the area lost to the insulation insert in the Hi-R-H and Hi-R blocks. Figures A-5 through A-8 provide in bar charts the numbers for the solid grouted blocks in Table A-3 for ease of comparison, while Figures A-9 through A-12 do the same for the numbers in Table A-4.

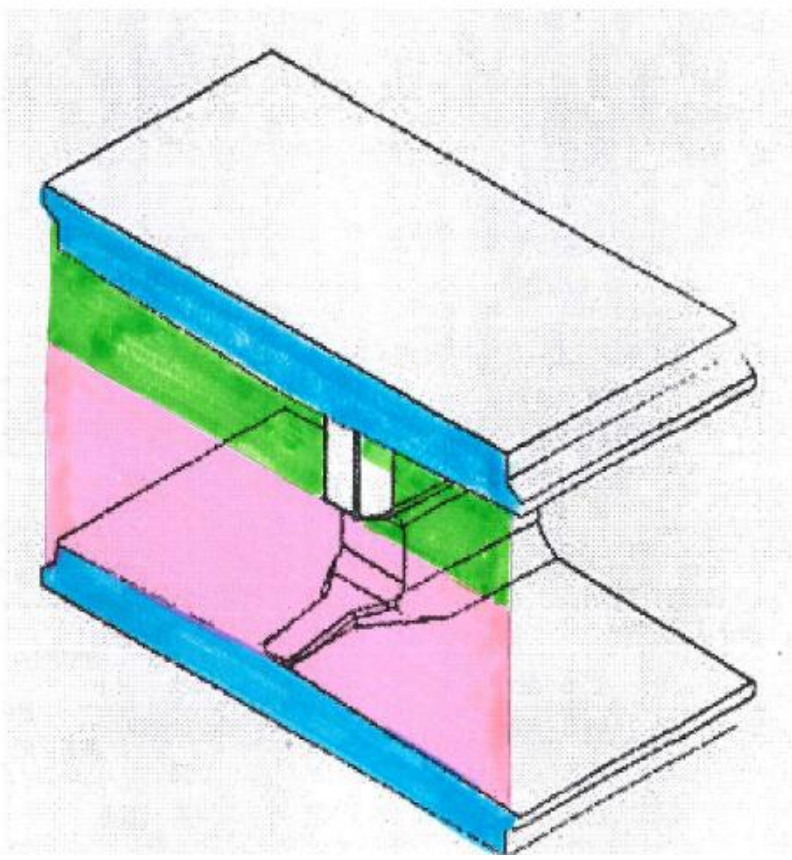
Unlike a standard block which is symmetric, the section modulus of the Hi-R-H and Hi-R blocks is different depending on the location of the insulation insert. The centroidal axis shifts towards the grouted side and away from the insulated side in the Hi-R-H and Hi-R blocks. Therefore, the section modulus is smaller for the insulation side than the grouted side of the Hi-R-H and Hi-R block.

A comparison of the bending moment-curvature relationship in Figure A-13 shows that the bending moment capacity of the Hi-R block is about 25% smaller than the standard block. The bending moment capacity of the Hi-R-H block is only 5% smaller than the Hi-R block. The moment-curvature computations for Figure A-13 assume a masonry compressive strength (f'_m) of 1500 psi (1900 psi block compressive strength) and a masonry modulus of elasticity (E_m) of 1350 ksi. The stress-strain curve of masonry in compression is represented by ACI’s Whitney stress block. The masonry is solid grouted. The vertical reinforcing steel is #4 bars located at 8” o.c. and has elastoplastic stress-strain curve with yield strength of 60 ksi. Vertical reinforcement is centered inside grouted area between insulation insert and face shell.

Figure A-14 illustrates the moment-curvature relationship for different compressive strengths of masonry, namely f'_m of 1500, 2000, and 2500 psi with E_m of 1350, 1800, and 2250 ksi, respectively. Figure A-14 shows that the Hi-R-H and the Hi-R blocks have slightly larger bending moment capacities for the larger masonry compressive strengths.

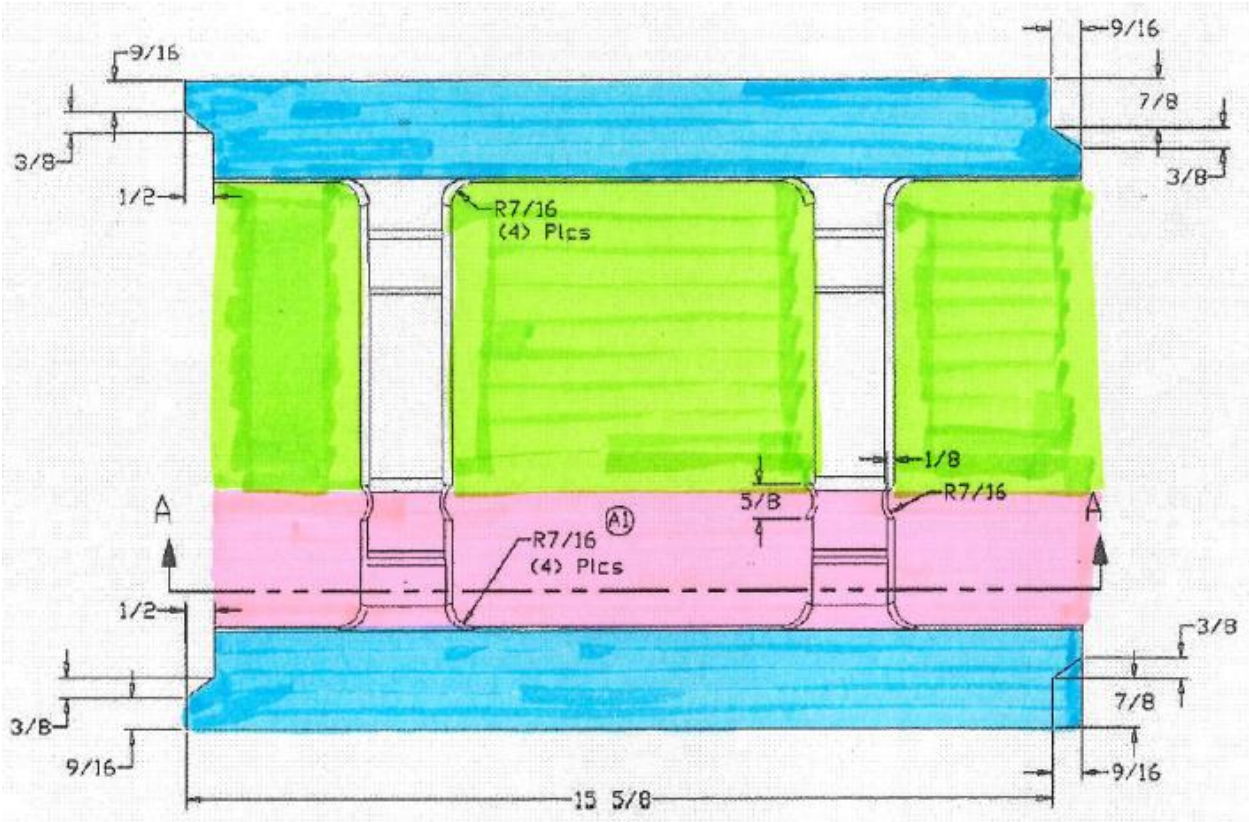
The above results show that the Hi-R-H and the Hi-R masonry blocks have comparable geometric and structural properties. The deviation in properties of the Hi-R-H block is at most within 7% of the Hi-R block.

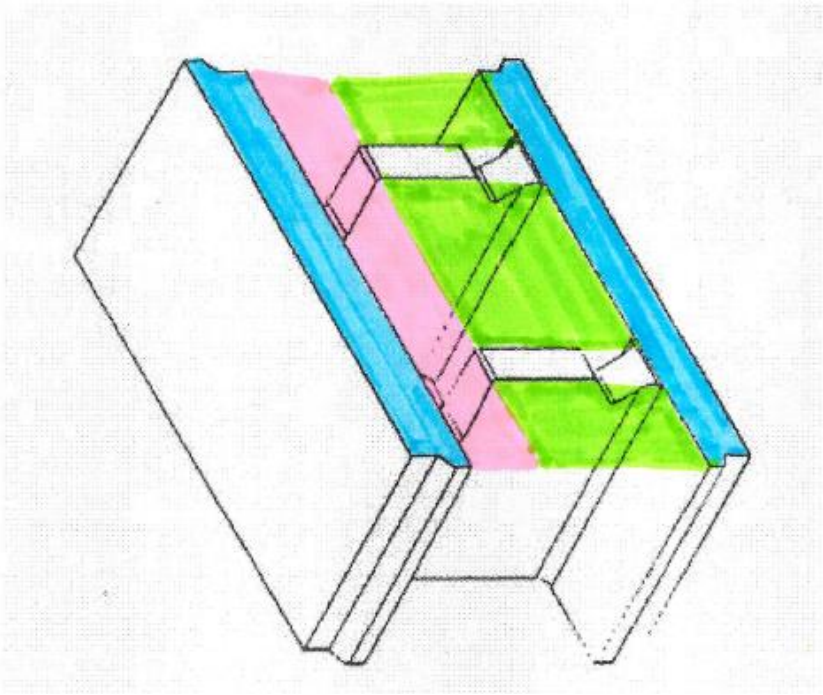




Blue: Face shell, **Green**: Grout, **Pink**: Insulation

Figure A-3 Hi-R-H 12" Block





Blue: Face shell, **Green**: Grout, **Pink**: Insulation

Figure A-4 Hi-R 12" Block

Table A-3 Comparison of Geometric Properties (12" Solid Grouted)

	Standard 12" Block	Hi-R 12" Block	Hi-R-H 12" Block	Difference between Hi-R-H and Hi-R
Cross Sectional Area (in2/ft)	140	108	104	3.7%
Moment of Inertia (in4/ft)	1571	1245	1178	5.4%
Section Modulus Insulation side (in3/ft)	270	188	174	7.4%
Section Modulus Grouted side (in3/ft)		249	243	2.4%

Table A-4 Comparison of Geometric Properties (12" and 10" Solid Grouted)

	Standard 12" Block	Hi-R 12" Block	Hi-R-H 12" Block	Standard 10" Block	Hi-R 10" Block	Hi-R-H 10" Block
Cross Sectional Area (in2/ft)	140	108	104	116	84	80
Moment of Inertia (in4/ft)	1571	1245	1178	892	700	716
Section Modulus Insulation side (in3/ft)	270	188	174	185	126	128
Section Modulus Grouted side (in3/ft)		249	243		172	177

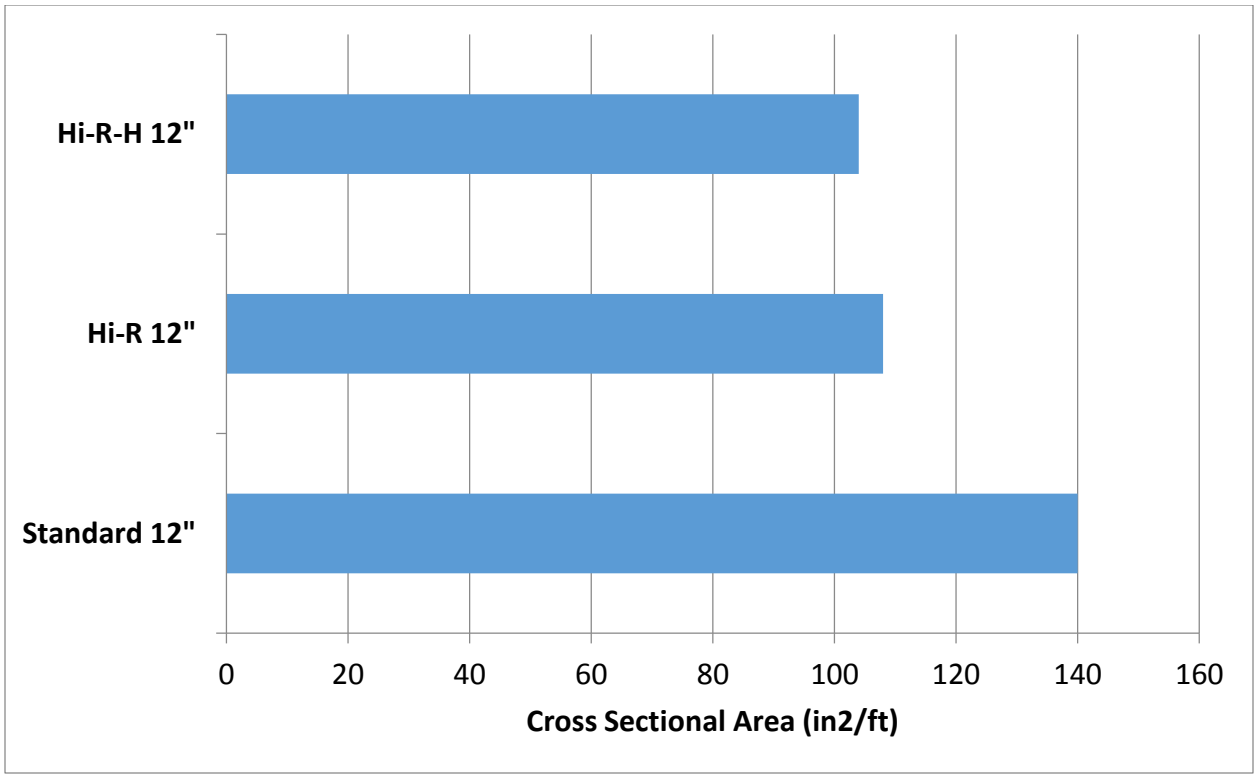


Figure A-5 Comparison of Cross Sectional Areas (12” Solid Grouted)

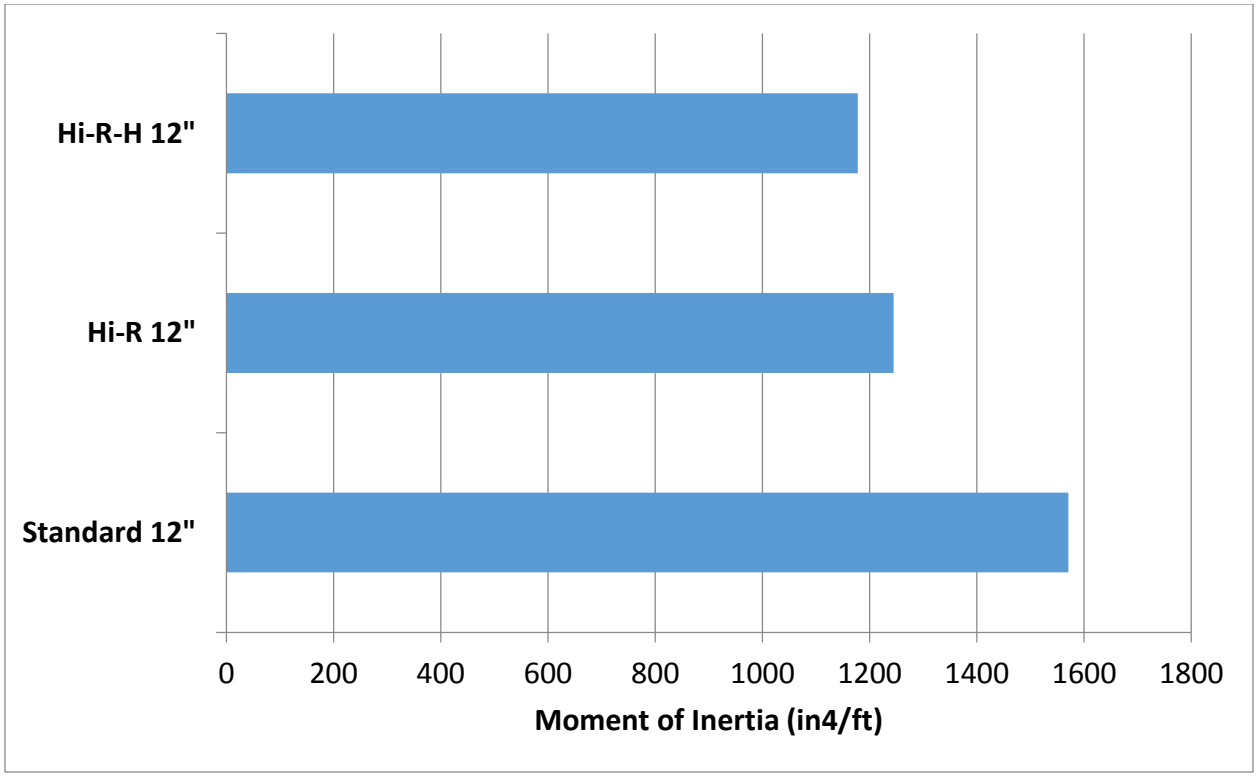


Figure A-6 Comparison of Moments of Inertia (12” Solid Grouted)

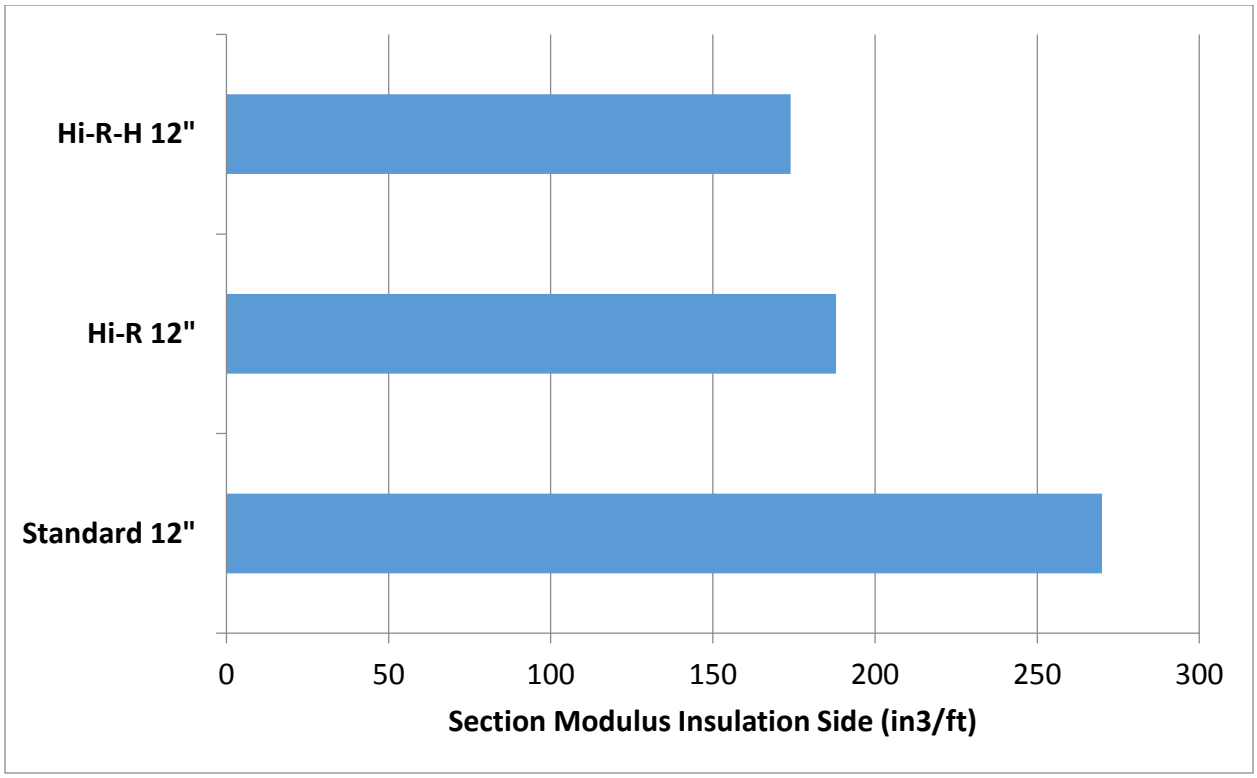


Figure A-7 Comparison of Section Moduli – Insulation Side (12” Solid Grouted)

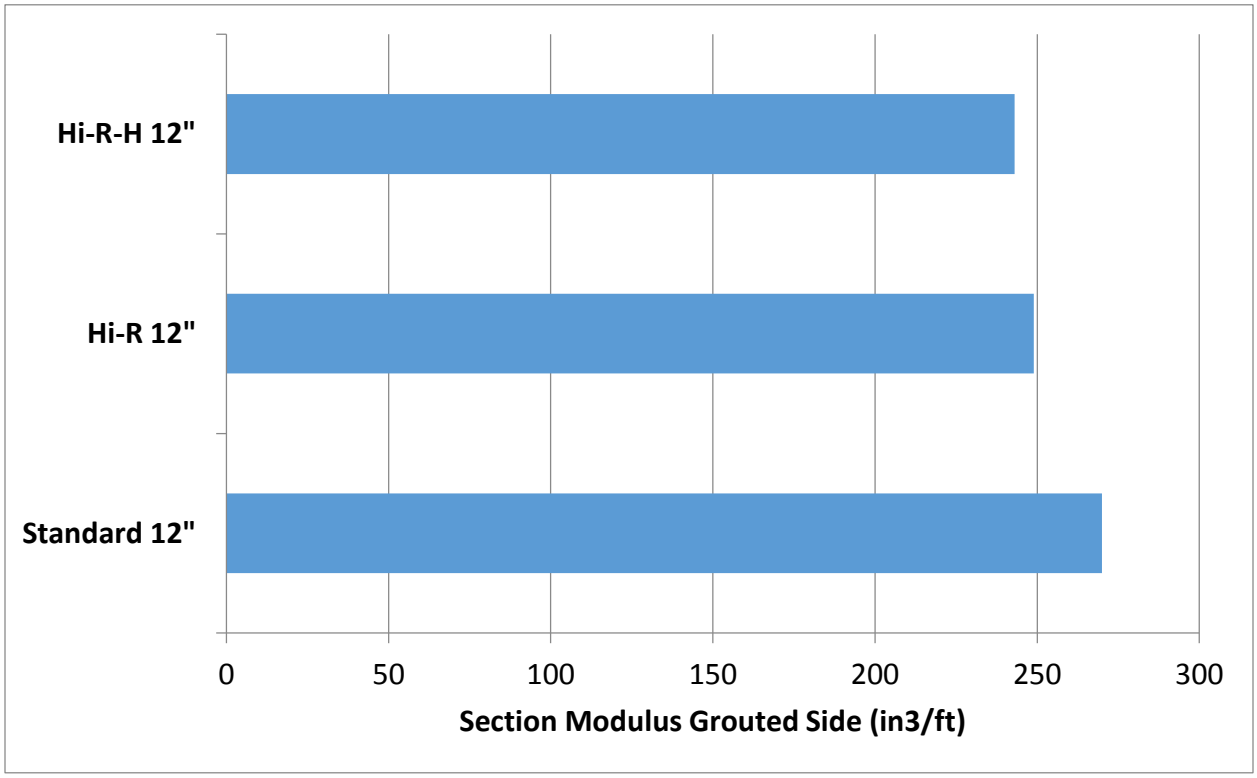


Figure A-8 Comparison of Section Moduli – Grouted Side (12” Solid Grouted)

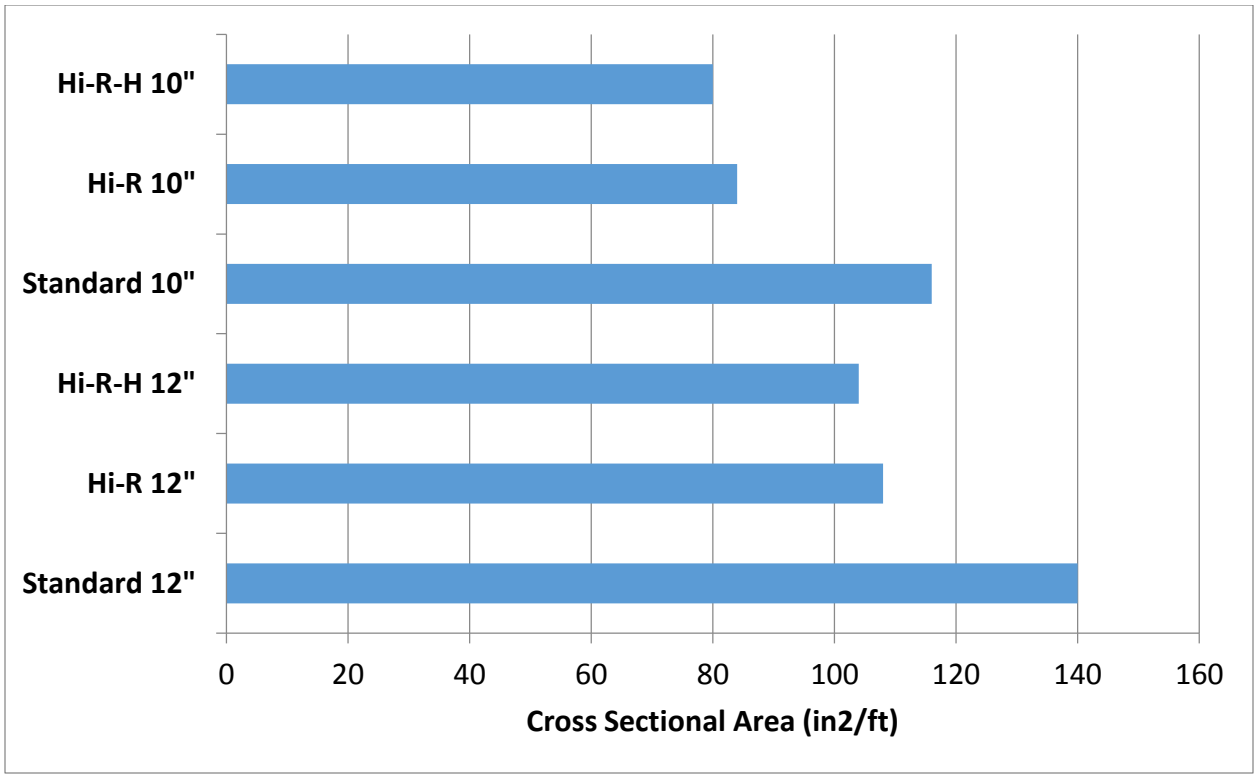


Figure A-9 Comparison of Cross Sectional Areas (10" and 12" Solid Grouted)

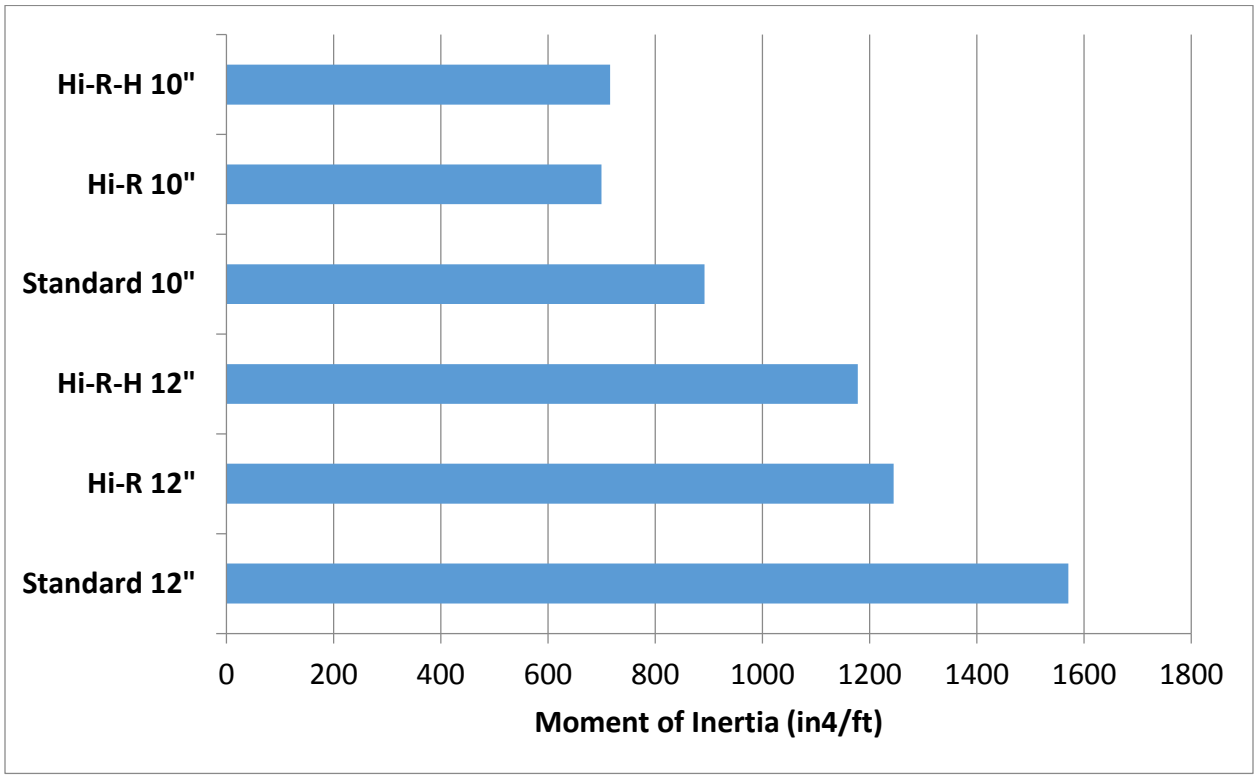


Figure A-10 Comparison of Moments of Inertia (10" and 12" Solid Grouted)

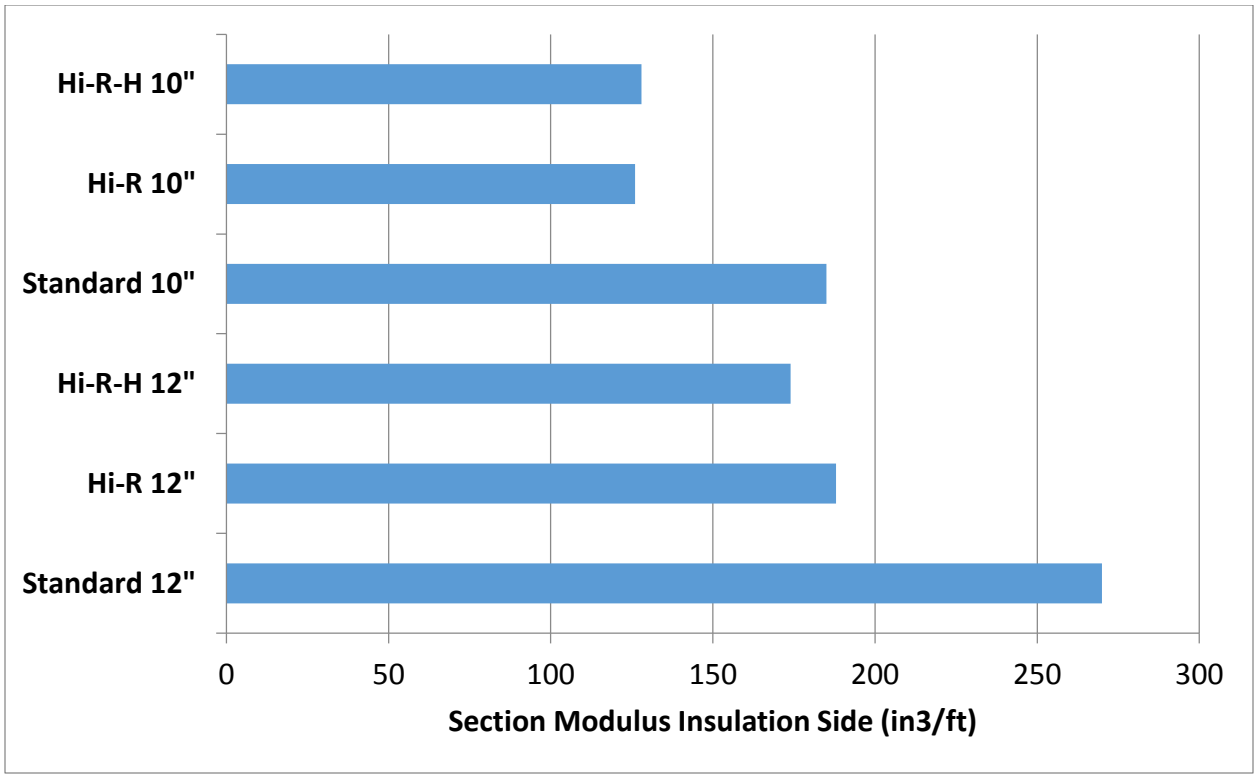


Figure A-11 Comparison of Section Moduli – Insulation Side (10" and 12" Solid Grouted)

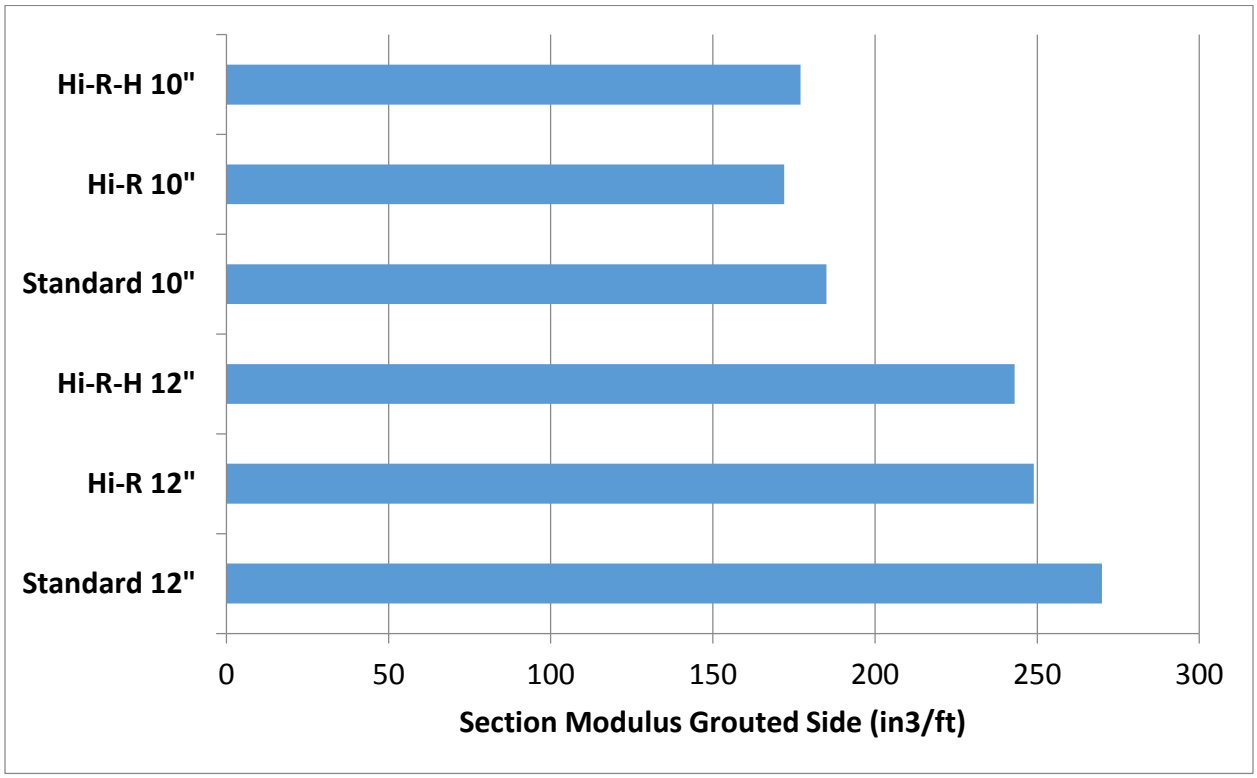


Figure A-12 Comparison of Section Moduli – Grouted Side (10" and 12" Solid Grouted)

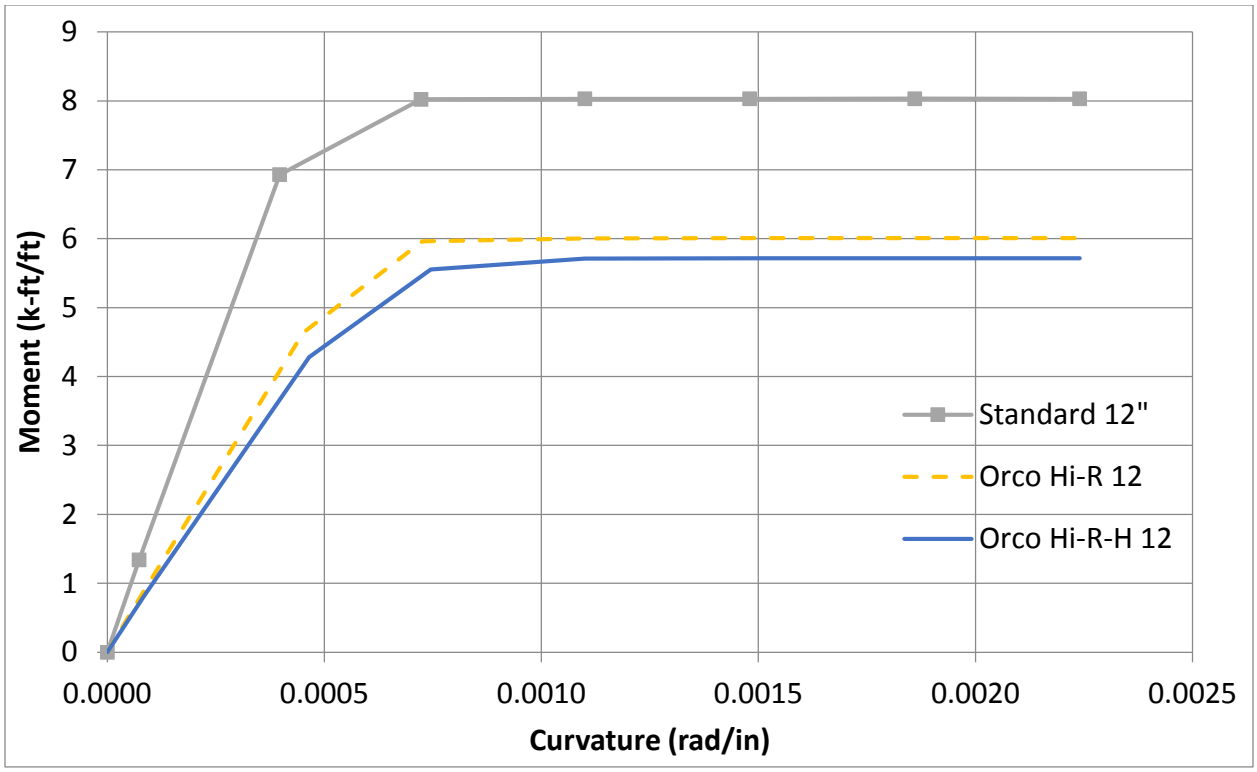


Figure A-13 Comparison of Moment-Curvature Curves for Different Blocks

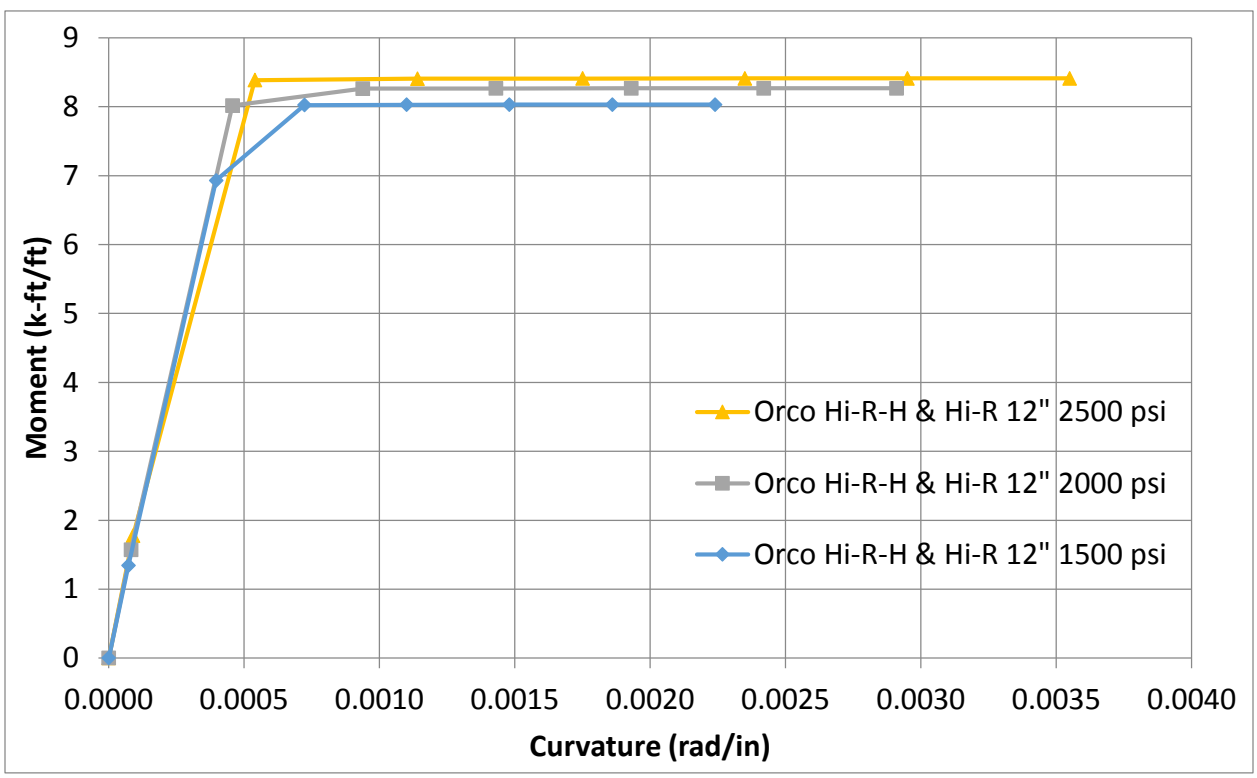


Figure A-14 Comparison of Moment-Curvature Curves for Different Masonry Strengths

APPENDIX B EXAMPLE DESIGNS OF SHEAR DOMINATED SHEAR WALLS

This section shows two examples of long shear walls. They are inspired from Example A.1 in the Design of Reinforced Masonry Structures (DORMS) book by Brandow, Ekwueme and Hart (2012). The two example walls are on Gridlines 1 and 3 as marked in the following figures from pages 225 to 232 in the DORMS book.

APPENDIX A - Building Design Examples

A.1 ONE-STORY WAREHOUSE

A one-story warehouse building is shown in Figures A1.1 and A1.2. The following elements of the building will be designed using strength design and allowable stress design procedures:

1. Determination of building earthquake loads.
2. Design of walls on gridline 1 to resist out-of-plane earthquake loads.
3. Design of wall to roof connection for walls on gridline 1.
4. Design of typical masonry pilaster on gridline A.
5. Design of lintel and jamb reinforcement around the 12-foot high openings on gridline G.

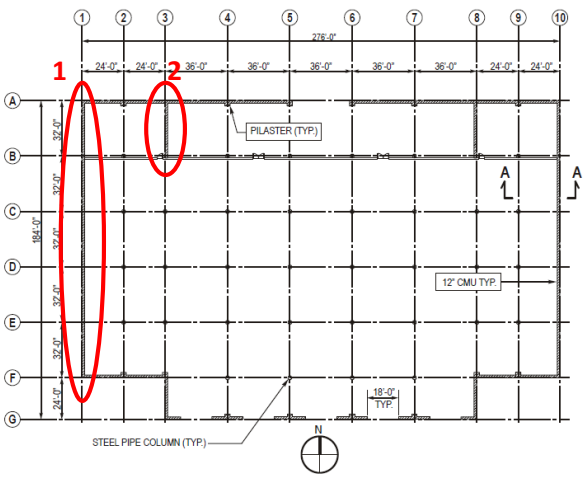


FIGURE A.1.1 Floor Plan of Example Building (Not to Scale)

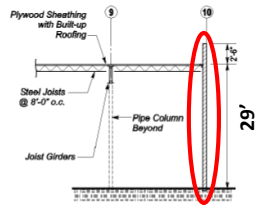


FIGURE A.1.2 Section A-A (Not to Scale)

A.1.1 Earthquake Loads on the Lateral Load Resisting System

Materials

Masonry:

12-inch thick, fully grouted, medium weight concrete masonry units (weight = 124 psf) laid in running bond. Masonry Compressive Strength, f'_m = 1500 psi, type S Mortar.

Steel:

Grade 60, E_s = 29,000ksi

Loads

Gravity Loads:

Roofing (Built-up)	=	5.0 psf
Sheathing (1/2" Plywood)	=	1.5
2x Rafters	=	1.0
Sprinklers	=	1.0
Mechanical Equipment	=	1.0
Insulation and misc.	=	1.5
		11.0 psf
Roof joists & girders	=	5.0
Total roof dead load	=	16.0 psf
Partitions		5.0
Total Seismic Load		21.0 psf
Roof live load (Reducible)	=	20 psf

Earthquake Loads:

Risk-targeted Maximum Considered Earthquake (MCE$_R$) 5% damped spectral response acceleration at short periods, S_S = 2.194g. MCE$_R$ 5% damped spectral response acceleration at 1-second period, S_1 = 0.739g. Site Class D, T_L = 8.0 seconds.

Seismic Base Shear

From Table 3.6.3, the site coefficients for Site Class D are given by:

$F_a = 1.0$

$F_v = 1.5$

The MCE$_R$ spectral response acceleration parameters adjusted for Site Class effects are therefore equal to:

$S_{MS} = F_a S_S = 1.0(2.194) = 2.194$ g

$S_{M1} = F_v S_1 = 1.5(0.739) = 1.11$ g

and the 5% damped, design spectral response values are given by:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(2.194) = 1.46 \text{ g}$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(1.11) = 0.74 \text{ g}$$

The control periods for the design response spectrum are obtained from Equations (3.6.9):

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.74}{1.46} = 0.10 \text{ sec}$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.74}{1.46} = 0.51 \text{ sec}$$

The seismic base shear will be calculated using the equivalent lateral force procedure.

The approximate fundamental period of the building is determined with Equation (3.7.9):

$$T_a = C_t h_n^x = 0.02(29)^{0.75} = 0.25 \text{ sec}$$

where C_t and x are obtained from Table 3.7.3. Note that since the building is a concrete masonry shear wall structure, the approximate fundamental period may also be determined from Equation (3.7.11).

From Tables 3.5.1 and 3.7.5, a warehouse, which is a Risk Category II structure, has a Seismic Importance Factor, I_e, of 1.0. The response modification factor, R, for special reinforced masonry shear walls (bearing wall system) is equal to 5 (Table 3.7.2). Therefore, the seismic response coefficient is equal to:

$$C_s = \frac{S_{DS} I_e}{R} = \frac{1.46(1.0)}{5} = 0.29$$

Since $T < T_L$, the seismic response coefficient need not be greater than:

$$C_s = \frac{S_{D1} I}{RT} = \frac{0.74(1.0)}{(5)(0.25)} = 0.59 > 0.29$$

The minimum base shear coefficient is given by the following equations:

$$V_{min} = 0.044 S_{DS} I_e W$$
$$= 0.044(1.46)(1.0)W = 0.064W < 0.29W$$

In addition, since S_1 is greater than 0.6g, the base shear must be no less than:

$$V = \frac{0.5 S_1 I_e}{R} W$$
$$= \frac{0.5(0.739)(1.0)}{5} W = 0.074W < 0.29W$$

Therefore the seismic base shear is equal to $0.29W$.

As permitted in Section 12.3.1.1 of ASCE 7-10, diaphragms constructed with wood structural panels may be idealized as flexible relative to the masonry walls. Thus, the earthquake loads are distributed in proportion to the tributary seismic weight supported by the masonry walls.

When designing buildings with flexible diaphragms, the seismic dead load on the diaphragm includes the weight of the roof and any permanently attached equipment, as well as the tributary weight (above mid-height for the one-story building) of the masonry walls that are normal to the direction of the earthquake load. The walls that are parallel to the direction of the earthquake load support the earthquake loads due to their own self-weight. As calculated earlier, the seismic weight of the roof is 21 psf (the roof live load is not included in the seismic weight). Therefore, for the walls in the east-west direction:

$$W_{walls} = 124\left(\frac{29}{2} + 2.5\right) = 2108 \text{ lbs/ft}$$

For the walls in the north-south direction, assuming 24-inch square pilasters spaced at 36 feet on center (pilasters do not extend to the parapet):

$$W_{walls} = 2108 + 124\left(\frac{29}{2}\right)\frac{2}{36} = 2208 \text{ lbs/ft}$$

Then, in the east-west direction between gridlines A and B (recalling that $C_S = 0.29$):

$$w = 0.29\left(\frac{21(276) + 2108(4)}{1000}\right) = 4.13 \text{ kips/ft}$$

Between gridlines B and F (276-foot length):

$$w = 0.29\left(\frac{21(276) + 2108(2)}{1000}\right) = 2.90 \text{ kips/ft}$$

Between gridlines F and G (180-foot length):

$$w = 0.29\left(\frac{21(180) + 2108(2)}{1000}\right) = 2.32 \text{ kips/ft}$$

In the north-south direction between gridlines 1 and 3 (as well as between gridlines 8 and 10):

$$w = 0.29\left(\frac{21(160) + 2208(2)}{1000}\right) = 2.26 \text{ kips/ft}$$

Between gridlines 3 and 8 (conservatively neglecting the openings):

$$w = 0.29\left(\frac{21(184) + 2208(2)}{1000}\right) = 2.40 \text{ kips/ft}$$

Figures A.1.3(a) and A1.3(b) show the diaphragm earthquake loads on the building in the east-west and north-south directions, respectively.

Diaphragm Shear Stresses and Chord Forces

The shear per unit length on the diaphragm is obtained by dividing the force by the appropriate length the diaphragm. To calculate the chord forces, the diaphragm is assumed to span as a simply supported beam between the collectors so that the moment in the diaphragm is given by:

$$M = \frac{wL^2}{8}$$

where L is the span between the collectors, and w is the earthquake load on the span. The chord force is then given by:

$$C = T = \frac{M}{D} = \frac{wL^2}{8D}$$

where D is the effective depth of the diaphragm parallel to the earthquake loads. For example, the chord force for the diaphragm span between gridlines 3 and 8 for forces in the north-south direction is given by:

$$C = T = \frac{wL^2}{8D} = \frac{2.40(180)^2}{8(184)} = 52.8 \text{ kips}$$

The steel required to resist this load is equal to:

$$A_{s,req'd} = \frac{T}{\phi f_y} = \frac{52.8}{0.9(60)} = 0.98 \text{ in}^2$$

Figures A.1.4(a) and A.1.4(b) show the diaphragm shears and chords forces for earthquake loads in the east-west and north-south directions, respectively.

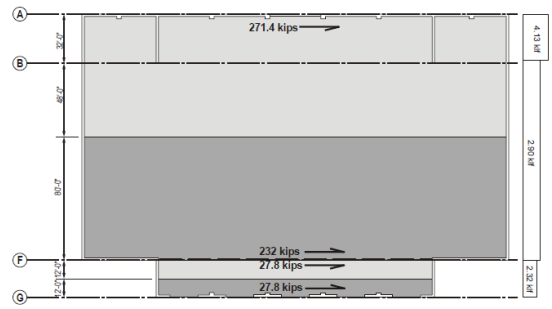


FIGURE A.1.3(a) Roof Diaphragm Earthquake Loads in East-West Direction

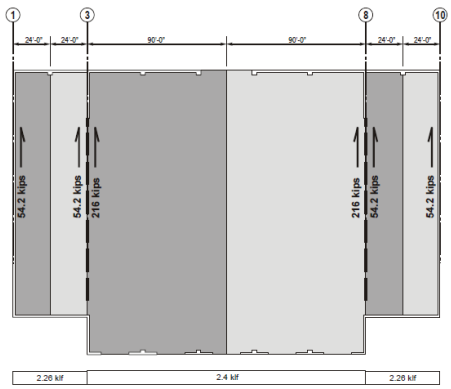


FIGURE A.1.3(b) Roof Diaphragm Earthquake Loads in North-South Direction

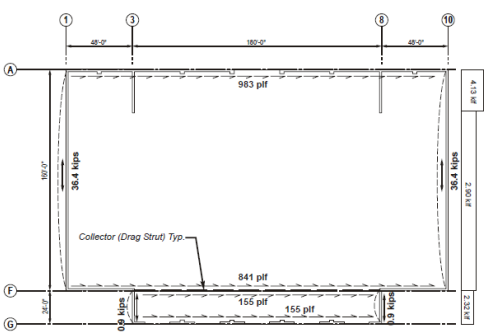


FIGURE A.1.4(a) Diaphragm Shear and Chord Forces for Earthquake Loads in the East-West Direction

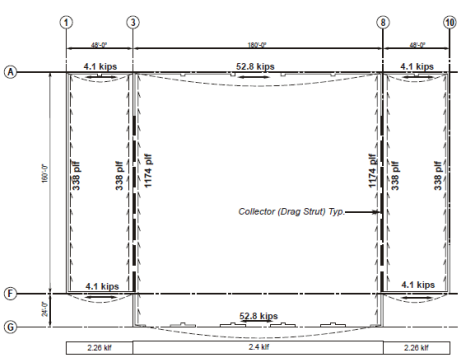


FIGURE A.1.4(b) Diaphragm Shear and Chord Forces for Earthquake Loads in the North-South Direction

In-plane Wall Forces

A portion of the earthquake loads on the shear walls are obtained by summing the diaphragm shears along each line of resistance. The weight of walls loaded in-plane was not included when calculating the shears for the flexible diaphragm. Therefore, the earthquake loads from self-weight of the walls must be added to the diaphragm shears to determine the shear at the base of the walls. Figure A.1.5 shows the forces on walls. For the two walls on either side of the opening on gridline A:

$$V_{dia} = \frac{271.4}{2} = 136 \text{ kips}$$

$$V_{wall} = 0.29(124)(31.5)\frac{276}{1000}\left(\frac{1}{2}\right)$$
$$= 1133\left(\frac{276}{1000}\right)\frac{1}{2}$$
$$= 156 \text{ kips}$$

$$V_{pilaster} = 0.29(124)(29)\left(\frac{2}{36}\right)\frac{276}{1000}\left(\frac{1}{2}\right)$$
$$= 58\left(\frac{276}{1000}\right)\frac{1}{2}$$
$$= 8 \text{ kips}$$

Therefore the shear on each wall is equal to:

$$V = 136 + 156 + 8 = 300 \text{ kips}$$

and the moment on each wall is given by:

$$M = 136(29) + 156\left(\frac{31.5}{2}\right) + 8\left(\frac{29}{2}\right) = 6517 \text{ kip-ft}$$

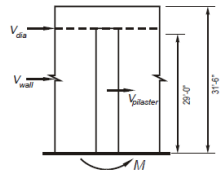


FIGURE A.1.5 Wall In-Plane Earthquake Loads

For the two walls on gridline F:

$$V = \frac{(232.3 + 27.8)}{2} + \frac{(1133)48}{1000} + \frac{(58)48}{1000}$$
$$= 130 + 54 + 3 = 187 \text{ kips}$$

$$M = 130(29) + 54\left(\frac{31.5}{2}\right) + 3\left(\frac{29}{2}\right) = 4664 \text{ kip-ft}$$

For the walls on gridlines 1 and 10:

$$V = 54.2 + \frac{1133(160)}{1000} = 54.2 + 181.3 = 236 \text{ kips}$$

$$M = 54.2(29) + 181.3\left(\frac{31.5}{2}\right) = 4427 \text{ kip-ft}$$

Lines 3 and 8 each contain two walls of different lengths. Therefore, the total diaphragm shear along the line of resistance must be distributed to the walls in proportion to their stiffness. The shear from the diaphragm along the line of resistance is equal to:

$$V = 54.2 + 216.1 = 270 \text{ kips}$$

Assuming there is no rotation of the foundation, the relative stiffness of the cantilever walls is given by Equation (3.8.9):

$$R_i = \frac{1}{\Delta_i} = \frac{1}{\left[4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]_i}$$

$$\Delta_1 = 4\left(\frac{29}{32}\right)^3 + 3\left(\frac{29}{32}\right) = 2.98 + 2.72 = 5.70$$

$$R_1 = \frac{1}{\Delta_1} = \frac{1}{5.70} = 0.175$$

$$\Delta_2 = 4\left(\frac{29}{24}\right)^3 + 3\left(\frac{29}{24}\right) = 7.06 + 3.63 = 10.69$$

$$R_2 = \frac{1}{\Delta_2} = \frac{1}{10.69} = 0.094$$

Thus the earthquake force from the diaphragm in each wall is given by:

$$F_1 = \frac{R_1}{\sum R} F = \left(\frac{0.175}{0.175 + 0.094}\right)270 = 176 \text{ kips}$$

$$F_2 = \frac{R_2}{\sum R} F = \left(\frac{0.094}{0.175 + 0.094}\right)270 = 94 \text{ kips}$$

The forces on the walls on gridlines 3 and 8 between gridlines A and B are given by:

$$V = 176 + \frac{1133(32)}{1000} = 176 + 36 = 212 \text{ kips}$$

$$M = 176(29) + 36\left(\frac{31.5}{2}\right) = 5671 \text{ kip-ft}$$

And for the walls on gridlines 3 and 8 between gridlines F and G:

$$V = 94 + \frac{1133(24)}{1000} = 94 + 27 = 121 \text{ kips}$$

$$M = 94(29) + 27\left(\frac{31.5}{2}\right) = 3151 \text{ kip-ft}$$

Collector (Drag Strut) Forces

The collectors or drag struts are elements that transfer lateral forces to the masonry shear walls that form the lateral force resisting system. As discussed in Section 3.7.2, the building code requires that collectors are designed to account for possible overstrength in the structure. This is achieved by using the special load combinations:

For allowable stress design:

$$(1.0 + 0.14S_{DS})D + 0.7\Omega_0 Q_E + H + F$$

$$(1.0 + 0.105S_{DS})D + 0.525\Omega_0 Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) + H + F$$

$$(0.6 - 0.14S_{DS})D + 0.7\Omega_0 Q_E + H$$

For strength design:

$$(1.2 + 0.2S_{DS})D + \Omega_0 Q_E + L + 0.2S$$

$$(0.9 - 0.2S_{DS})D + \Omega_0 Q_E + 1.6H$$

where Q_E is the horizontal component of the earthquake load and Ω_0 is the overstrength factor in Table 3.7.2.

The collector force can be calculated from the difference between the shear in the diaphragm and the shear in the wall. Therefore in the east-west direction along gridline F, the shear per unit length on each wall due to diaphragm earthquake loads is equal to:

$$V = \frac{130(1000)}{48} = 2708 \text{ lbs/ft}$$

Therefore the force in the collector at gridline 3 is equal to:

$$V = \frac{(841 - 2708)}{1000} 48 = -89.6 \text{ kips}$$

and the collector force at gridline 8 is equal to:

$$V = -89.6 + \frac{(841 + 155)180}{1000} = 89.6 \text{ kips}$$

Similarly, for the collectors on gridlines 3 and 8, the collector force at gridline F is equal to:

$$V = \frac{1174(24)}{1000} - 94 = -65.8 \text{ kips}$$

and the collector force at gridline B is equal to:

$$V = -65.8 + \frac{(1174 + 338)128}{1000} = 127.7 \text{ kips}$$

The collector forces are shown in Figures A.1.6(a) and (b). The forces must be multiplied by the overstrength coefficient Ω_0 and the appropriate load combinations used when designing the collector.

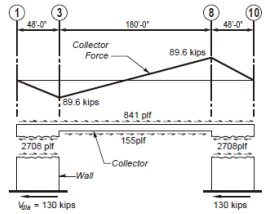


FIGURE A.1.6(a) Collector Forces Along Gridline F

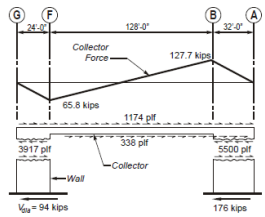


FIGURE A.1.6(b) Collector Forces Along Gridlines 3 and 8

Determination of Redundancy Factor, ρ

For concrete masonry structures, the redundancy factor, ρ, may be taken as 1.0 if the following conditions are satisfied:

- Removal of any shear wall or pier with a height to length ratio greater than 1.0 does not result in a loss of story shear of more than 33 percent, or create an extreme torsional irregularity.
- The structure is regular in plan at all levels and the seismic lateral load resisting system consists of at least two perimeter shear walls on each side of the structure in each orthogonal direction with a length equal to or greater than two times the story height.

The building is regular and by inspection, the removal of any 29-foot length of wall does not result in a loss in shear strength of more than 33 percent. Therefore, the redundancy factor, ρ = 1.0.

A.1.2 Design of Wall on Gridline 1 to Resist Out-of-Plane Loads

Gravity Loads

Since the joist girders are 24 feet away from the wall, the roof width tributary to wall is equal to 0.5(24) =12 ft. Then:

Roof dead load	=	16(12)	=	192 lbs/ft
Roof live load	=	20(12)	=	240 lbs/ft
Self-weight of wall at mid-height:				
Parapet	=	124(2.5)	=	310 lbs/ft
Wall	=	124(29)(0.5)	=	1798
				2108 lbs/ft

Thus the axial loads at the critical wall cross-section, which occurs at the wall mid-height, are equal to:

P_{fD} = 192 lbs/ft

P_{fLr} = 240 lbs/ft

P_w = 2108 lbs/ft

Figure A.1.7 shows the connection of the roof to the wall. From the figure the eccentricity of the roof reaction, which occurs at the edge of the 4x ledger is given by:

$$e = \frac{t}{2} + 3.5 = \frac{11.63}{2} + 3.5 = 9.3 \text{ in}$$

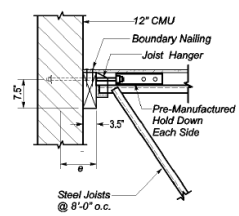


FIGURE A.1.7 Wall Connection to Roof

Seismic Loads

From Equation (3.9.2), the out-of-plane earthquake load on the structural wall is equal to:

$$F_p = 0.4S_{DS}I_p W_p$$
$$= 0.4(1.46)(1.0)(124) = 72.4 \text{ psf}$$

Strength Design

From Section 3.2.1, the load combinations that need to be considered for earthquake design are as follows:

$$1.2D + E$$

$$0.9D + E$$

Taking into account the vertical component of earthquake load, the load combinations become:

1. $(1.2 + 0.2 \times 1.46)D + E_h = 1.49D + E_h$
2. $(0.9 - 0.2 \times 1.46)D + E_h = 0.61D + E_h$

For load combination 1, the axial loads on the wall are given by:

$$P_{uf} = 1.49(192) = 268 \text{ lbs/ft}$$

$$P_{uw} = 1.49(2108) = 3141 \text{ lbs/ft}$$

$$P_u = P_{uf} + P_{uw} = 268 + 3141 = 3409 \text{ lbs/ft}$$

$$\frac{P_u}{A_g} = \frac{3409}{12(11.63)} = 24.4 \text{ psi} \le 0.05 f'_m \quad \text{...OK}$$

Try 2 layers of #5 bars spaced at 16 inches center. Only the layer with bars in tension is considered since the reinforcement is not laterally supported to resists compression. (A_s = 0.23 in^2 /ft, d = 9.2 in) and determine the wall properties.

$$I_g = \frac{b(t)^3}{12} = \frac{12(11.63)^3}{12} = 1573 \text{ in}^4 \text{ /ft}$$

From Table 1.6.2 the modulus of rupture, f_r, for solid-grouted masonry in running bond with Type S mortar and flexural stresses normal to bed joints is 163 psi. The cracking moment is thus given by:

$$M_{cr} = \left(f_r + \frac{P_u}{A_g}\right) S_n = (163 + 24.4)\frac{12(11.63)^2}{6}$$
$$= 50694 \text{ lbs-in/ft} = 4225 \text{ lbs-ft /ft}$$

To determine the cracked moment of inertia, I_{cr}, the location of the neutral axis is obtained from Section 4.5.1:

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} = \frac{0.23(60,000) + 3409}{0.64(1500)(12)} = 1.49 \text{ in}$$

In the above figures, the long wall on gridline 1 is marked as Example 1, and the long wall on gridline 3 is marked as Example 2. Example 1 wall has an aspect ratio of a little over 5, while Example 2 wall has an aspect ratio of a little over 1. The two long walls are designed for in-plane shear using 12” Hi-R blocks as follows:

EXAMPLE 1 WALL (GRIDLINE 1)

Step #1 Is the Shear Wall a Long Wall?

Height of the Shear Wall = *h* = 31.5'

Length of Shear Wall = ℓ = 160'

Length to Height Ratio = (ℓ/*h*) = 5.08

Conclusion: Yes this is a Long Wall.

Step #2 Calculate Earthquake Design Load (Factored Load) on the Wall

R = 5 (Special Reinforced Masonry Wall)

V_1 = Base Shear Force = 236 kips (See box in the figures above)

M_1 = Base Overturning Moment = 4427 kip-ft (See box in the figures above)

TMS 402-13 (M/Vd) Ratio = $\left(M_1 / V_1 d \right)$

d = TMS 402-13 notation for length of shear wall = ℓ = 160'

$\left(M_1 / V_1 d \right)$ = 0.12

Conclusion: This is a TMS 402-13 defined Long Shear Wall because $\left(M_1 / V_1 d \right)$ is less than 1.0.

Step #3 Verify that Shear Wall Moment Capacity is Sufficient to Classify Wall as a Long Shear Wall

Vertical Reinforcing Steel = #4 bars @ 16 inches on center

Specified (Nominal) Reinforced Steel Bar Stress = f_y = 60,000 psi

Specified (Nominal) Compressive Strength = $f_m^{'}$ = 1,500 psi

Nominal Moment Limit State Capacity of the wall= M_n = 140,000 kip-ft for the axial load demand

Limit State Capacity / Demand Check = $\left(M_n / M_1 \right)$ = 140,000 / 4427 = 31 > 1.0 OK

Check shear corresponding to $1.25M_n > \emptyset V_n$ [*see $\emptyset V_n$ calculation in Step #4*]

($1.25M_nV_1/M_1$) = 1.25*140,000*236/4427 = 9330 kips > 0.7*3120 = 2184 kips OK

Conclusion: Moment capacity sufficient to classify wall as long shear wall.

Step #4 Calculate the Limit State Nominal Capacity of Wall

Nominal Shear Limit State Capacity of the wall = $V_n = V_m + V_s$ = 2640 + 480 = 3120 kips

(This is calculated using masonry area of 108 in^2/ft for Hi-R per ESR-3508 Table 1, and #4@24" horizontal steel)

Verify that Nominal shear limit state capacity of wall (V_n = 3120 kips) < $6A_{nv}\sqrt{f'_m}$ = 4020 kips OK

Capacity Reduction Factor = ϕ = 0.7

(This value of 0.7 was calculated using structural reliability analysis as outlined in Appendix E, where C_{TEST} = 101 kips, α_1= 1.28, B = 4.0, C =2.0, V_t = 0.10 per test data summarized in Appendix F)

Limit state capacity / demand check = $\left(\phi V_n / V_1\right)$ = 0.7 x 3120 / 236 = 9.3 > 1.0 OK

Nominal shear limit state capacity / Design demand = (V_n / V_u) = (V_n / 2.5V_1) = 5.3 > 1 OK

Conclusion: The design of the Long Wall is acceptable.

Note that Example 1 wall is a very long wall with an aspect ratio of 5 and therefore the shear strength provided by the masonry alone is sufficient to resist 2.5 times the shear force demand on the wall. As such, minimum horizontal reinforcement is provided (#4@24").

Horizontal reinforcement provided (#4@24") has steel area of 0.0009 times the Hi-R masonry area, which is greater than 0.0007 to satisfy TMS 402-13 Section 7.3.2.6. OK

Vertical reinforcement provided (#4@16") has steel area of 0.0014 times the Hi-R masonry area, which is greater than 0.0007 to satisfy TMS 402-13 Section 7.3.2.6. OK

Horizontal and vertical reinforcement combined has steel area of 0.0023 times the Hi-R masonry area, which is greater than 0.002 to satisfy TMS 402-13 Section 7.3.2.6. OK

Vertical reinforcement provided has steel area of 0.0014 times the Hi-R masonry area, which is greater than one-third of horizontal steel area of 0.0003 to satisfy TMS 402-13 Section 7.3.2.6. OK

EXAMPLE 2 WALL (GRIDLINE 3)

Step #1 Is the Shear Wall a Long Wall?

Height of the Shear Wall = *h* = 31.5'

Length of Shear Wall = ℓ = 32'

Length to Height Ratio = (ℓ/*h*) = 1.02

Conclusion: Yes this is a Long Wall.

Step #2 Calculate Earthquake Design Load (Factored Load) on the Wall

R = 5 (Special Reinforced Masonry Wall)

V_1 = Base Shear Force = 212 kips (See box in the figures above)

M_1 = Base Overturning Moment = 5671 kip-ft (See box in the figures above)

TMS 402-13 (M/Vd) Ratio = $\left(M_1 / V_1 d \right)$

d = TMS 402-13 notation for length of shear wall = ℓ = 32'

$\left(M_1 / V_1 d \right)$ = 0.84

Conclusion: This is a TMS 402-13 defined Long Shear Wall because $\left(M_1 / V_1 d \right)$ is less than 1.0.

Step #3 Verify that Shear Wall Moment Capacity is Sufficient to Classify Wall as a Long Shear Wall

Vertical Reinforcing Steel = #5 bars @ 16 inches on center

Specified (Nominal) Reinforced Steel Bar Stress = f_y = 60,000 psi

Specified (Nominal) Compressive Strength = f'_m = 1,500 psi

Nominal Moment Limit State Capacity of the wall = M_n = 7900 kip-ft for the axial load demand

Limit State Capacity / Demand Check = $\left(M_n / M_1 \right)$ = 7900 / 5671 = 1.4 > 1.0 OK

Check shear corresponding to $1.25M_n > \emptyset V_n$ [*see $\emptyset V_n$ calculation in Step #4*]

($1.25M_nV_1/M_1$) = 1.25*7900*212/5671 = 371 kips > 0.7*530 = 370 kips OK

Conclusion: Moment capacity sufficient to classify wall as long shear wall.

Step #4 Calculate the Limit State Nominal Capacity of Wall

Nominal Shear Limit State Capacity of the wall = $V_n = V_m + V_s$ = 360 + 170 = 530 kips

(This is calculated using masonry area of 108 in^2/ft for Hi-R per ESR-3508 Table 1, and #4@24" + #3@16" horizontal steel)

Verify that Nominal shear limit state capacity of wall (V_n = 530 kips) < $4.4A_{nv}\sqrt{f'_m}$ = 590 kips OK

Capacity Reduction Factor = ϕ = 0.7

(This value of 0.7 was calculated using structural reliability analysis as outlined in Appendix E, where C_{TEST} = 101 kips, α_1= 1.28, B = 4.0, C =2.0, V_t = 0.10 per test data summarized in Appendix F)

Limit state capacity / demand check = $\left(\phi V_n / V_1\right)$ = 0.7 x 530 / 212 = 1.8 > 1.0 OK

Nominal shear limit state capacity / Design demand = (V_n / V_u) = (V_n / 2.5V_1) = 1.01 > 1 OK

Conclusion: The design of the Long Wall is acceptable.

Horizontal reinforcement provided (#4@24" + #3@16") has steel area of 0.0017 times the Hi-R masonry area, which is greater than 0.0007 to satisfy TMS 402-13 Section 7.3.2.6. OK

Vertical reinforcement provided (#5@16") has steel area of 0.0022 times the Hi-R masonry area, which is greater than 0.0007 to satisfy TMS 402-13 Section 7.3.2.6. OK

Horizontal and vertical reinforcement combined has steel area of 0.0039 times the Hi-R masonry area, which is greater than 0.002 to satisfy TMS 402-13 Section 7.3.2.6. OK

Vertical reinforcement provided has steel area of 0.0022 times the Hi-R masonry area, which is greater than one-third of horizontal steel area of 0.0006 to satisfy TMS 402-13 Section 7.3.2.6. OK

APPENDIX C COMPARISON OF DESIGN PROCEDURE WITH IBC AND TMS

Strength design of concrete block masonry was first developed and submitted to ICC ES in 1982 and was approved in 1983. The strength design shear capacity equation was first adopted by the Uniform Building Code (UBC) in 1984.

The IBC 2015 is the latest edition of ICC and uses the TMS strength design shear capacity equation.

From TMS 402-13, recall Section 7.3.2.6.1 *Shear capacity design*.

> 7.3.2.6.1.1 "When designing special reinforced masonry shear walls to resist in-plane forces in accordance with Section 9.3, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u."

When using the 2015 IBC, the R value for a special reinforced masonry shear wall is used for strength design; it is R = 5. However, the earthquake shear force on a Long Wall is multiplied by 2.5. This is equivalent to using an R value of 2. This amplified earthquake shear force demand corresponds to 0.4 times R=5.

TMS 402-13 Code Commentary:
7.3.2 *Participating elements* – "A seismic-force-resisting system must be defined for every structure. Most masonry buildings use masonry shear walls to serve as the seismic-force-resisting system, although, other systems are sometimes used (such as concrete or steel frames with masonry infill). Such shear walls must be designed by the engineered methods in Part 3, unless the structure is assigned to Seismic Design Category A, in which case empirical provisions of Appendix A may be used.

Twelve shear wall types are defined by the Code. Depending upon the masonry material and detailing method used to design the shear wall, each wall type is intended to have a different capacity for inelastic response and energy dissipation in the event of a seismic event. These twelve shear wall types are assigned system design parameters such as response modification factors, *R*, based on their expected performance and ductility. Certain shear wall types are permitted in each seismic design category, and unreinforced shear wall types are not permitted in regions of intermediate and high seismic risk. Table CC-7.3.2-1 summarizes the requirements of each of the twelve types of masonry shear walls."

TABLE CC-7.3.2-1 Requirements for Masonry Shear Walls Based on Shear Wall Designation[1]

Shear Wall Designation	Design Methods	Reinforcement Requirements	Permitted In
Empirical Design of Masonry Shear Walls	Section A.3	None	SDC A
Ordinary Plain (Unreinforced) Masonry Shear Walls	Section 8.2 or Section 9.2	None	SDC A and B
Detailed Plain (Unreinforced) Masonry Shear Walls	Section 8.2 or Section 9.2	Section 7.3.2.3.1	SDC A and B
Ordinary Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.3.1	SDC A, B, and C
Intermediate Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.5	SDC A, B, and C
Special Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.6	SDC A, B, C, D, E, and F
Ordinary Plain (Unreinforced) AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.7.1	SDC A and B
Detailed Plain (Unreinforced) AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.8.1	SDC A and B
Ordinary Reinforced AAC Masonry Shear Walls	Section 11.3	Section 7.3.2.9	SDC A, B, C, D, E, and F
Ordinary Plain (Unreinforced) Prestressed Masonry Shear Walls	Chapter 10	None	SDC A and B
Intermediate Reinforced Prestressed Masonry Shear Walls	Chapter 10	Section 7.3.2.11	SDC A, B, and C
Special Reinforced Prestressed Masonry Shear Walls	Chapter 10	Section 7.3.2.12	SDC A, B, C, D, E, and F

[1] Section and Chapter references in this table refer to Code Sections and Chapters.

From the TMS 402-13, recall Section 7.3.2.6 *Special reinforced masonry shear walls* – "Design of special reinforced masonry shear walls shall comply with the requirements of Section 8.3, Section 9.3, or Appendix C. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.3.1 and the following:

(a) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(b) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be uniformly distributed, shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall, and shall be embedded in grout. The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(c) The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

1 For masonry laid in running bond, the minimum cross-sectional area of reinforcement in each direction shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

2 For masonry not laid in running bond, the minimum cross-sectional area of vertical reinforcement shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions. The minimum cross-sectional area of horizontal reinforcement

shall be at least 0.0015 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

(d) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

(e) Mechanical splices in flexural reinforcement in plastic hinge zones shall develop the specified tensile strength of the spliced bar.

(f) Masonry not laid in running bond shall be fully grouted and shall be constructed of hollow open-end units or two wythes of solid units."

Table C-1 also provides a comparison between the design procedure here and the TMS requirements.

Table C-1 Comparison of Requirements and Restrictions with TMS 402-13

	Hi-R, Hi-R-H Evaluation	**TMS 402-13**
Design Methodology	Strength Design only	Allowable Stress Design or Strength Design
Masonry Compressive Tests	Verify strength and strain	Verify strength only
Grouting	Fully grouted only	Partially or fully grouted
Mortar	Type S mortar only	--
Width of Masonry Units	10" or 12"	--
Units Layout Pattern	Running bond only	Stack or running bond
Wall Geometry	Rectangular walls only	--
Reinforcement Restrictions	#8 or less	#9 or less
Peer Review	Required	--
Structural Observation	Required	--

APPENDIX D SHEAR WALL CAPACITY AND DEFORMATION

The following is based on TMS 402-13 Section 9.3.6.4 Shear Strength of reinforced masonry shear wall design for in-plane loads, with references to TMS 402-13 Sections 9.3.4.1.2 and 7.3.2.6.1.1. The equations are directly from TMS 402-13:

> "The shear strength of shear walls is determined using the same equations used for the design of beams columns and piers. However, in order to minimize the possibility of brittle shear failures in shear walls located in high seismic zones, shear demands are calculated differently. TMS 402-13 stipulates that the design shear strength ϕV_n must exceed the shear corresponding to 1.25 times the nominal flexural strength of the wall. In addition, the nominal shear strength need not exceed 2.5 times the shear demand obtained from the structural analysis.
>
> The contribution of both masonry and steel may be used to calculate shear strength:
>
> $$V_n = V_{nm} + V_{ns} \tag{D.1}$$
>
> where:
>
> $$V_{nm} = \left[4 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u \tag{D.2}$$
>
> and
>
> $$V_{ns} = 0.5 A_v f_y \frac{d}{s} \tag{D.3}$$
>
> In Equation (D.3) the shear reinforcement is considered only fifty percent effective when the contribution of masonry is included in the calculation of shear strength. The shear strength must not exceed the following values:
>
> When $M_u/(V_u d_v) \leq 0.25$:
>
> $$V_n \leq 6 A_n \sqrt{f'_m} \tag{D.4}$$
>
> and when $M_u/(V_u d_v) \geq 1.0$:
>
> $$V_n \leq 4 A_n \sqrt{f'_m} \tag{D.5}$$
>
> The maximum value of V_n for $M_u/(V_u d_v)$ between 0.25 and 1.0 shall be permitted to be linearly interpolated."

Now consider, the deflection of a wall is calculated as the summation of flexural and shear deflection, see Figure D-1. The shear deflection is

$$\Delta_s = \frac{1.2VH}{AE_v}$$

with

A = L t

and therefore it follows that

$$\Delta_s = \left(\frac{1.2VH}{tL}\right) = \left(\frac{1.2V}{t}\right)\left(H / L\right)$$

The flexural deflection is

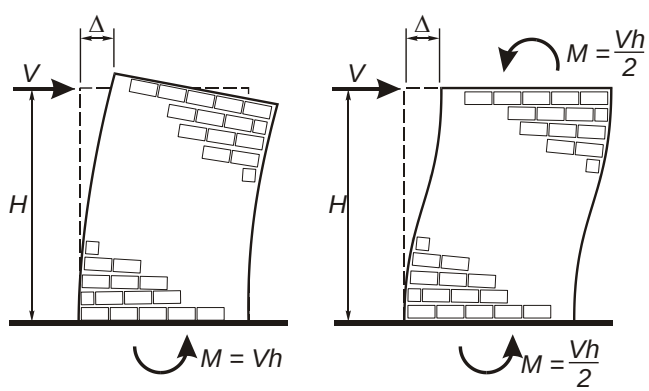
$$\Delta_f = \frac{VH^3}{12E_m I} \quad \text{for “fixed-fixed” walls or piers}$$

with

$$I = \frac{1}{12}\left(tL^3\right)$$

It follows that

$$\Delta_f = \frac{V\left(12\right)}{12E_m t}\left(\frac{H}{L}\right)^3$$



(a) Cantilever Wall/Pier (b) Fixed-Fixed Pier

FIGURE D-1 Shear Wall Deformation (DORMS 2012, Figure 3.8.9)

$$\Delta = \Delta_f + \Delta_s = \frac{VH^3}{12E_m I} + \frac{1.2VH}{AE_v}$$

where Δ_f and Δ_s are the wall deflections due to flexural and shear deformations, respectively. As with cantilever shear walls, the effect of foundation rotation is neglected to simplify the calculations. Noting that $E_v = 0.4E_m$ and substituting above equations for the wall area and moment of inertia:

$$\Delta = \left[\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]\left(\frac{V}{tE_m}\right)$$

and neglecting the common terms, the relative rigidity of "fixed-fixed" walls or piers is given by:

$$R = \frac{1}{\Delta} = \frac{1}{\left[\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]}$$

APPENDIX E DESIGN PROCEDURE DEVELOPMENT USING STRUCTURAL RELIABILITY THEORY

Figure E-1 shows the role of horizontal and vertical reinforcing steel in the basic deformation modes shear and flexural. The Squat Wall will have shear deformation and thus the reinforcing steel provides significant capacity and will be seen later.

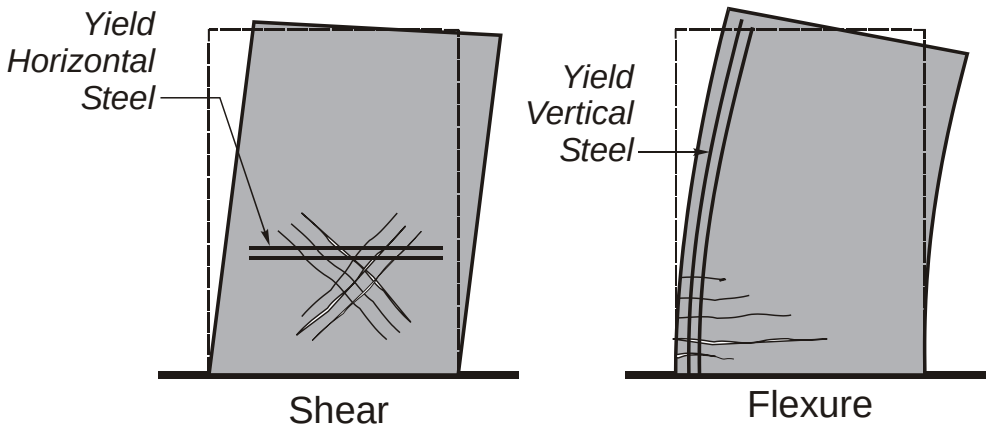


FIGURE E-1 Deformation Modes and Failure Patterns of Masonry Shear Walls (DORMS, Figure 3.8.1)

The development of the design procedure followed this two-step approach.

STEP 1: THE TEST DATA

Using test data we can determine the maximum tested capacity for each test and the Expected (Mean) Value of these maximum capacities.

Define

$$\bar{C}_{TEST} = \text{Expected (Mean) Value of the Tests} \qquad \text{(E1.1)}$$

The code equation for Nominal Shear Capacity given in TMS 402-13 Section 9.3.4.1.2 is:

$$\bar{C}_{TMS,EQ} = V_{nm} + V_{ns} \qquad \text{for fully grouted walls} \qquad \text{(E1.2)}$$

The ratio of the test capacity, $\bar{C}_{TEST}$, to this code equation with nominal material values is

$$\alpha_1 = \left(\bar{C}_{TEST} / \bar{C}_{TMS,EQ}\right) \geq 1.0 \qquad \text{(E1.3)}$$

STEP 2: THE DETERMINATION OF THE VALUE OF THE CAPACITY REDUCTION FACTOR

The Design Equation for capacity using Structural Reliability Theory is:

Design Capacity = $\hat{\phi}\bar{C}_{TEST}$ (E2.1)

$$\hat{\phi} = \exp\left[-0.75\beta\rho_C\right] \quad \text{(E2.2)}$$

β = Structural Reliability (Safety) Index

ρ_C = Coefficient of Variation of Capacity

Substituting Equation (E1.3) into Equation (E2.1), we obtain

Design Capacity = $\left(\hat{\phi}\alpha_1\right)\bar{C}_{TMS,EQ}$ (E2.3)

Now define a Capacity Reduction Factor as

$\phi = \hat{\phi}\alpha_1$ ≤ 0.8 [this is the limit set per TMS 402-13 for shear] (E2.4)

it follows that

Design Capacity = $\phi\bar{C}_{TMS,EQ}$ (E2.5)

ϕ is to be used for Hi-R and Hi-R-H shear walls in lieu of Ø = 0.8 for shear in TMS 402-13 Section 9.1.4.5.

Figure E-2 shows a plot of ϕ versus α_1 and ρ_C for β = 4.

Note the following for ρ_C :

$\rho_C = V = CV_t$

where

V_t = Coefficient of Variation of the maximum strength ≥ 0.10

where

$C = 2.0 - 0.1n \geq 1.0$

n = Number of replicate specimens

The value of *n* for this equation is 0 if less than 5 specimens are tested.

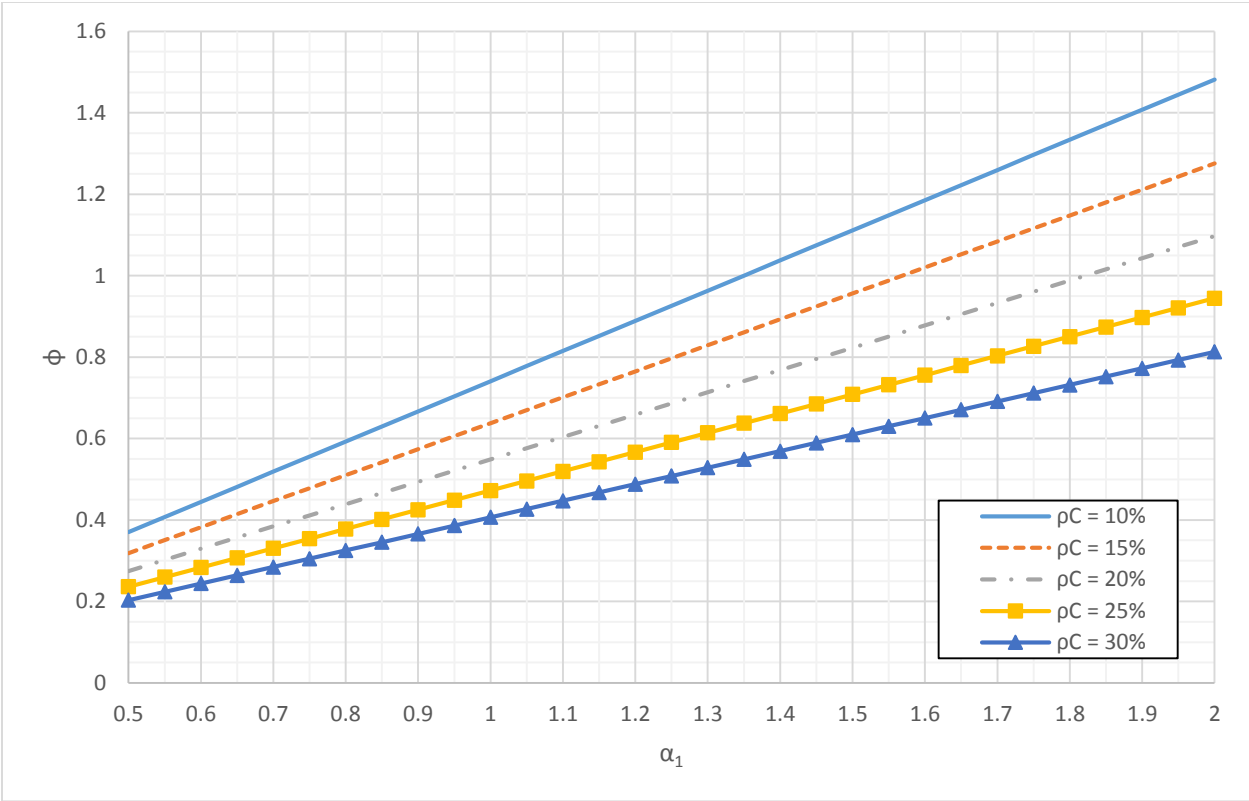


FIGURE E-2 Variation of Capacity Reduction Factor Ø with α_1 and ρ_c for $\beta = 4$

APPENDIX F MASONRY TEST DATA WITH KORFIL INSERTS

RESULTS OF FULLY GROUTED MASONRY PRISM TESTS (WITH AND WITHOUT INSULATION INSERTS)

1986 NCMA Report pages 42-44, 62
The following prism strength
f'_m values are tabulated in psi.

	WITHOUT INSERTS		WITH INSERTS
Specimen E.1.1	2220	Specimen E.4.1	2085
Specimen E.1.2	2550	Specimen E.4.2	2160
Specimen E.1.3	2200	Specimen E.4.3	1660
Mean	2323	Mean	1968
Std. Dev.	197	Std. Dev.	270
COV	8.5%	COV	13.7%

Notes:

1. Presence of inserts reduces the mean prism strength to 85% of its value without the inserts.
2. The COV is acceptable, i.e. less than 16% reported by Yokel et al. (1970) for 8-inch regular concrete block units.

A comparison of geometric and structural properties of masonry walls with regular blocks versus Hi-R blocks and Hi-R-H blocks, as well as other test data (axial, flexural, shear) is also available in Weidlinger report no. 14-05 dated June 25, 2014, which was submitted as part of evidence for ESR-3508 issued in March 2015. Part of this information is also reproduced in Appendix A of this report.

RESULTS OF FULLY GROUTED SHEAR WALL TESTS
(WITH INSULATION INSERTS)

Figure F-1 shows the shear test specimen.

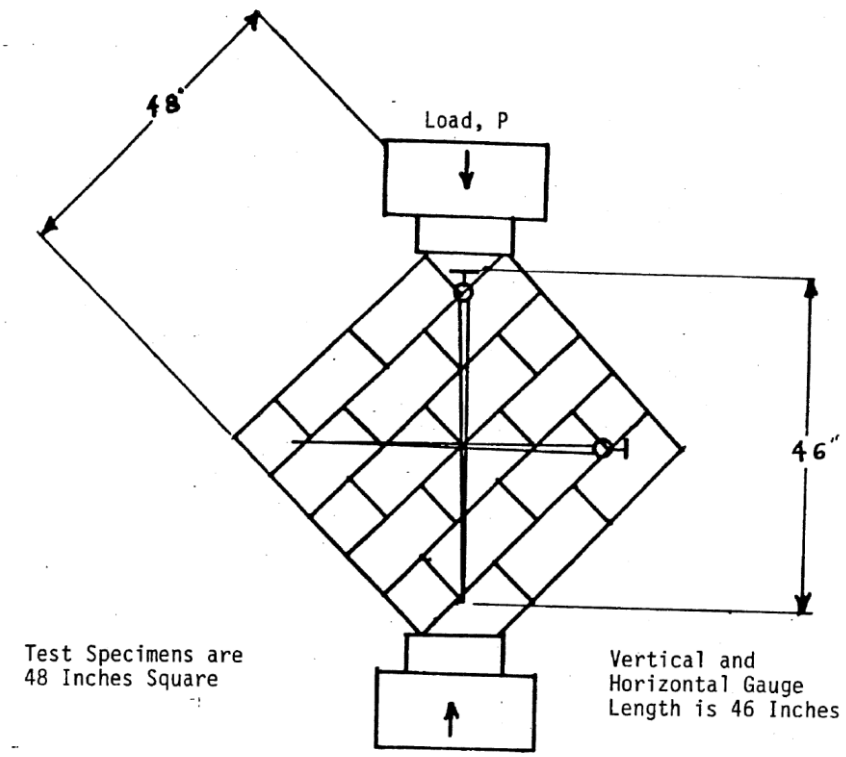


Figure F-1 Shear Testing of Walls

Three structural members were tested to determine the maximum shear strength of the fully grouted Long Wall with insulation inserts. The tests were monotonic shear tests. The load versus deflection curve for the tests are shown in Figure F-2.

Constructed masonry long walls have been field tested in past earthquakes successfully when they were designed using current TMS 402 or the prior UBC design equations.

Table F-1 Test data versus shear capacity in TMS 402-13 (kips)

	Average of 3 tests	Vn	ØVn*
Wall shear test with full grouting, insulation inserts, and #4@24"	100.7	79.2	63.4

*Ø = 0.8 for shear per TMS 402-13 (Note Ø = 0.7 for shear as recommended in this report)

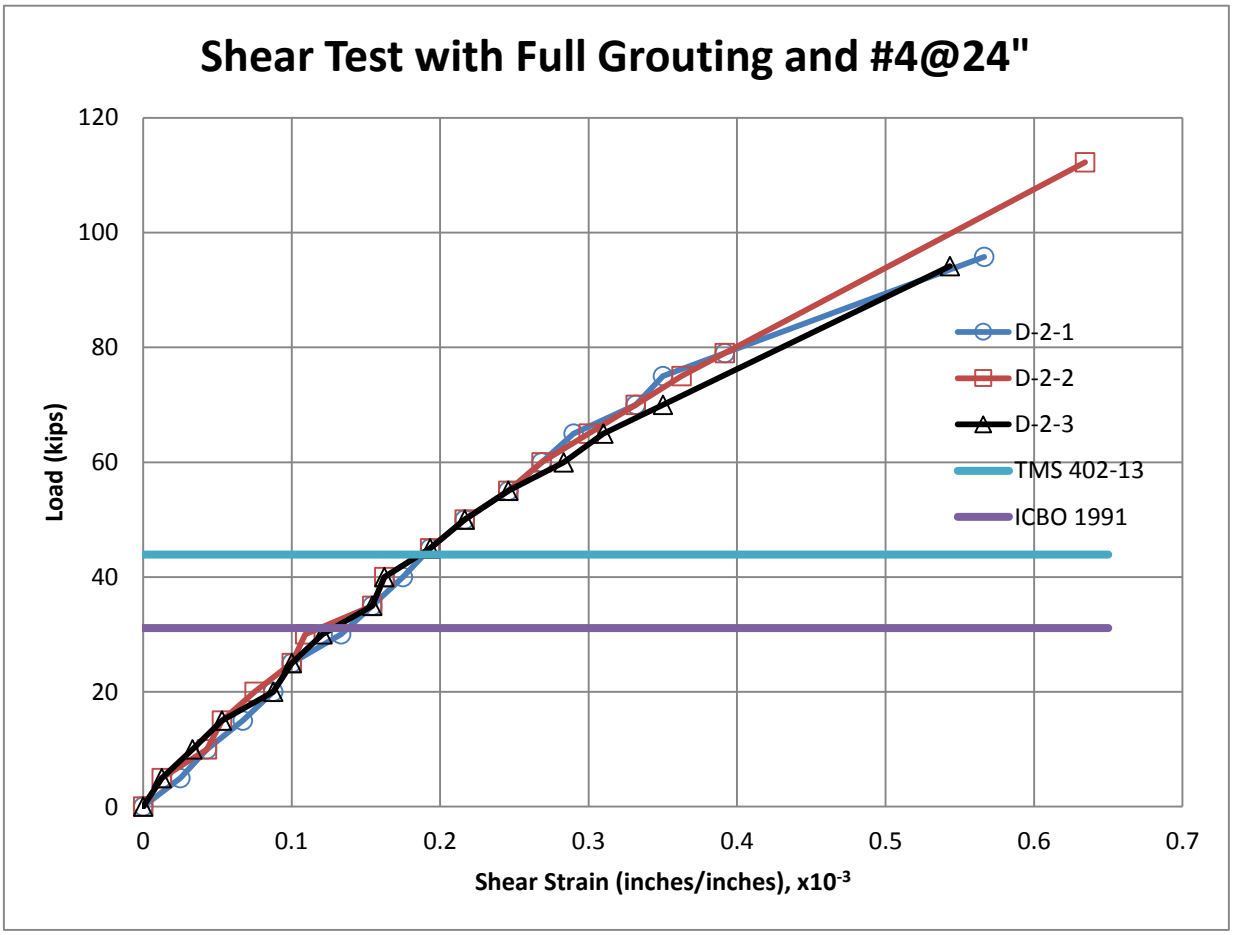


Figure F-2 Load versus shear strain from wall shear tests (wall with #4 bars at 24" o.c., insulation inserts, and full grouting)

DISCUSSION OF CYCLIC LOAD REVERSAL TESTING

Consider the following definition from (ATC-72-1, pg. B-1):

> **Cyclic Deterioration:** Hysteretic reduction in strength, stiffness, or both, as a function of the damage and energy dissipated in a yielding component. [Underline by Hart]

Structural deterioration does not start until the yielding of a wall and a Long Wall is not designed to experience significant yielding, if any, and that is why the R = 5 is used in conjunction with the seismic shear demand multiplied by 2.5 (i.e. in effect it is an R value of 2 because of this multiplication) and this R value represents a value that does not count on any ductility for the Long Wall. The Long Wall design procedure defines a shear limit state which exceeds the basis for the requirements in the IBC 2015 for the shear limit state which defines the Ultimate Deformation to correspond to a location on the descending branch of the load versus deflection curve where the load has dropped 20% from the maximum load. Consider the definitions from codes and standards in Appendix A.

For Long Walls and the shear limit state we expect very little difference between the capacity of the wall for a given horizontal displacement and the stabilized cyclic capacity at that same horizontal wall displacement.

The capacity of the long wall test data is at the first diagonal crack. This corresponds to typically the first kink in the load test curve. Several diagonal cracks develop later along the test curve and one of the diagonal cracks eventually widens into a principal diagonal tension crack.

Because the capacity is limited to the first kink in the load test curve and there is no degradation prior to this point. In support of this, the following excerpts and explanations are general definitions and concepts on static and cyclic behavior and cyclic degradation. They are taken from the introductory chapters of ATC-72-1 and FEMA P440A and not from material specific chapters. In other words, they are true irrespective of the type of construction material being tested and also true for its modified version as proprietary material.

PEER/ATC 72-1, p.2-17: Initial backbone curve obtained from a cyclic test is equivalent to monotonic loading curve obtained from a static test. “In concept, the differences between the initial backbone curve and the monotonic loading curve are small, and the terms initial and monotonic are interchangeable for practical purposes.” The difference between the two is virtually none when prior to yield, no cyclic deterioration takes place.

PEER/ATC 72-1, p.2-21: Four modes of cyclic deterioration are shown in Figure 2-12: Basic strength deterioration, post-capping strength deterioration, unloading stiffness deterioration, and accelerated reloading stiffness deterioration. Note that none of these modes causes any deterioration or change of response prior to first yield.

PEER/ATC 72-1, p.2-24: “Except at small deformations, the cyclic envelope (skeleton) curve falls clearly below the monotonic loading curve.” In other words, at small deformations, the cyclic envelope and the monotonic loading curves are essentially the same. This is clearly seen in Figure 2-16.

FEMA P440A, p.2-15: Figure 2-15 shows degradation occurring only post-yield and post-peak (inward movement of the capacity curve). In some cases of steel, this curve may move outward. However, in no case, the pre-yield portion of the curve changes.

FEMA P440A, p.2-21: Under lateral displacements that are less than or equal to those used to generate the cyclic envelope, differences between the cyclic envelope and the force-displacement capacity boundary are of no consequence. However, under larger lateral displacements these differences will affect the potential for in-cycle degradation to occur, which will significantly affect system behavior and response.

FEMA P440A, p.2-17, p.2-18, p.2-19: The choice of cyclic loading protocol has an effect on the cyclic envelope curve as shown in Figures 2-17 through 2-20, but note that it does not affect the load capacity at small displacements prior to first yield.

Figure F-3 taken from Chapter 3 of FEMA 307 show test results for reinforced concrete masonry walls failing in shear (Shing et al. 1991). Note that the cyclic response causes a strength drop of about 15% from the backbone envelope curve, but only after the specimen has yielded and the diagonal crack has occurred. Prior to this non-ductile failure in shear, the specimen is essentially linear elastic with little or no degradation in strength or stiffness. Compare this to a wall specimen failing in flexure with fat hysteretic loops and large drop in strength during cycles at large displacements or ductilities (Figures F-4 and F-5). A long masonry shear wall (with an effective R value of 5/2.5=2 per code) never reaches this level of ductility that a short shear wall does in flexure, and therefore it does not experience the corresponding cyclic strength loss although it fails in brittle mode.

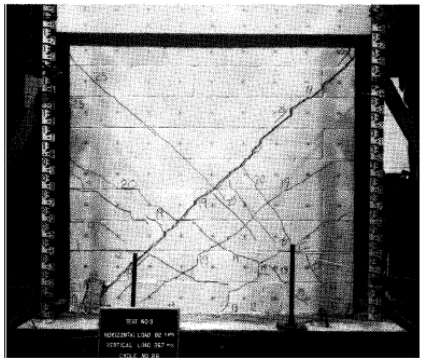
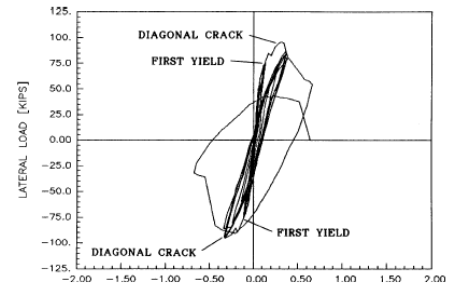
RM1 or RM2 Preemptive Shear *See Guide RM2G*	Shing et al., 1991 Specimen 9		

Table 3-1 Damage Patterns and Hysteretic Response for Reinforced Masonry Components (continued)

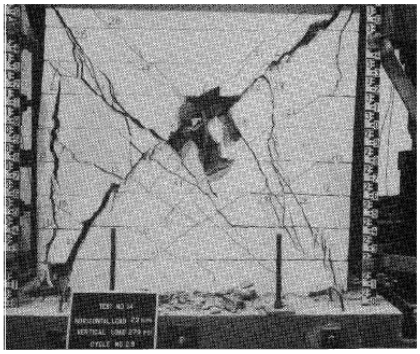
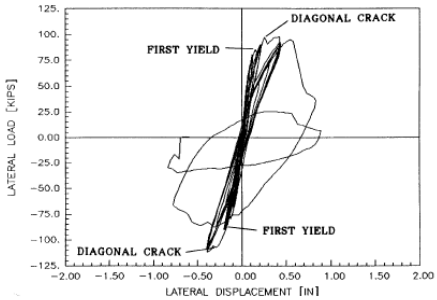
RM1 or RM2 Preemptive Shear *See Guide RM2G*	Shing et al., 1991 Specimen 14		

FIGURE F-3 Cyclic Test Data on Walls with Brittle Shear Failure (FEMA 307 Table 3-1)

Table 3-1 Damage Patterns and Hysteretic Response for Reinforced Masonry Components

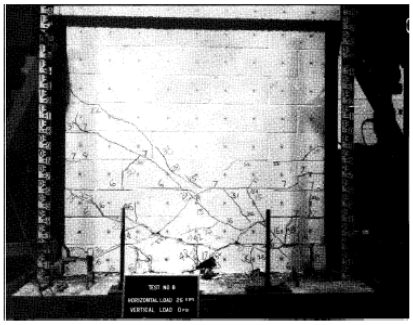
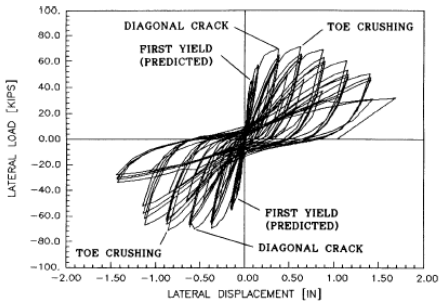
Component and Behavior Mode	Reference	Crack / Damage Pattern	Hysteretic Response
RM1 Flexure *See Guide RM1A*	Shing et al., 1991 Specimen 12		

FIGURE F-4 Cyclic Test Data on a Wall with Ductile Flexural Failure (FEMA 307 Table 3-1)

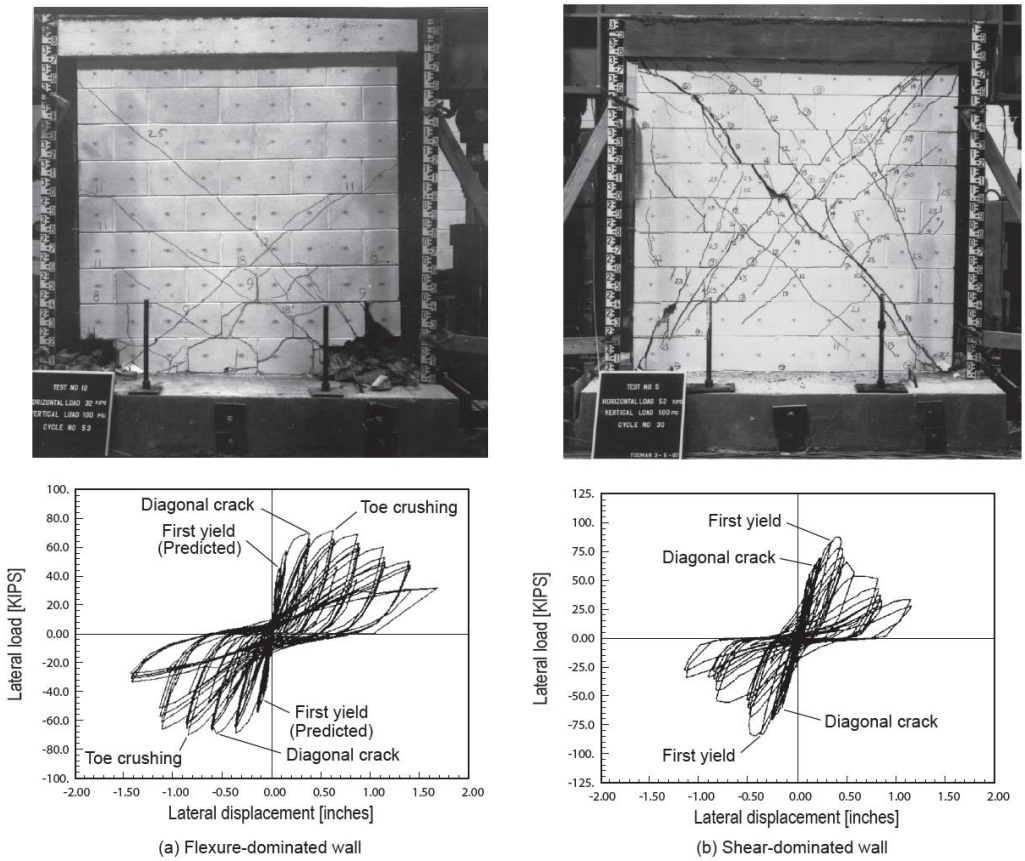


Figure 3-2. Behavior of flexure-dominated and shear-dominated walls (Shing et al. 1989).

FIGURE F-5 Comparison of shear-dominated wall to flexure-dominated wall - Reproduced from NIST GCR 14-917-31 (NEHRP Seismic Design Technical Brief No. 9 - Seismic Design of Special Reinforced Masonry Shear Walls)

APPENDIX G VERIFICATION OF DESIGN PROCEDURE USING METHODOLOGY IN FEMA P695/ATC 63

In order to verify the design procedure outlined in this report, the methodology and acceptance criteria of FEMA P695/ATC 63 was used. An overview and evaluation of the FEMA P695 methodology is also presented in the NIST GCR 10-917-8 report (2010). 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. 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. ASCE 7-10 Table C.1.3.1b (reproduced below as Table G-1) lists the acceptable probabilities of failure for earthquake loads for various risk categories of structures.

Table G-1 ASCE 7-10 Maximum Probabilities of Failure

Table C.1.3.1b Anticipated reliability (maximum probability of failure) for earthquake[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

[1]Refer to the NEHRP Recommended Provisions Seismic Regulation for Buildings and Other Strucures, FEMA P750, for discussion of the basis of seismic reliabiltiies.

The following tasks were performed to verify that the seismic design of Hi-R and Hi-R-H walls described in this report meets the maximum probability of collapse criteria of ASCE 7-10:

- The capacity curves for the reinforced masonry walls described in Examples 1 and 2 of Appendix B were obtained from a nonlinear static (pushover) analysis using the FEM/I Software (1987, Reformatted 2000). These are shown in Figures G-1 and G-2.

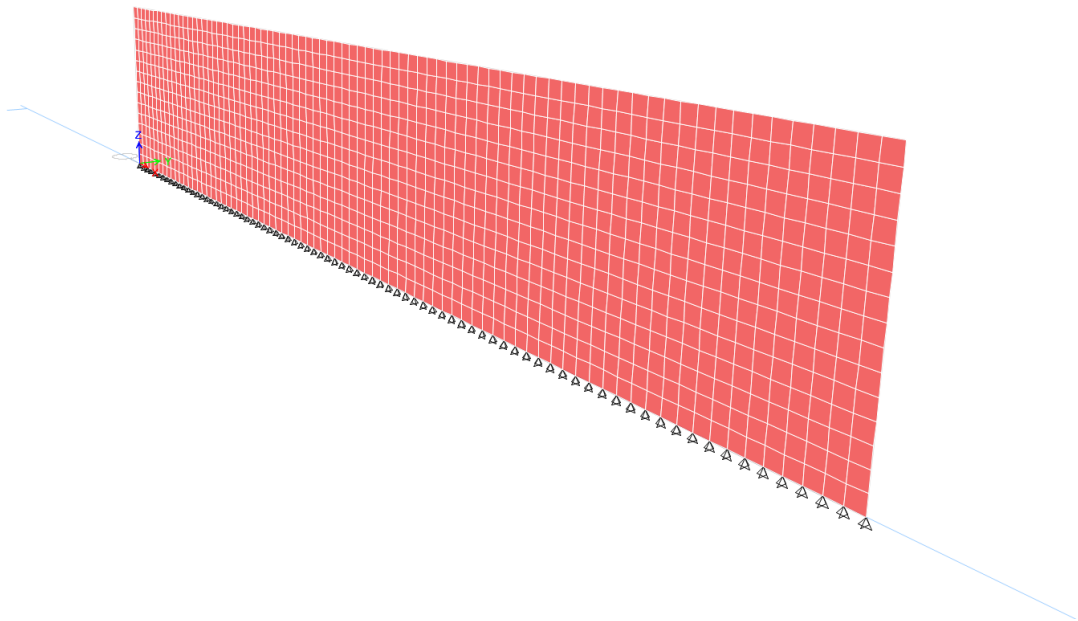


Figure G-1 Wall 1 (Example 1) Finite Element Mesh and Capacity Curve from FEM/I

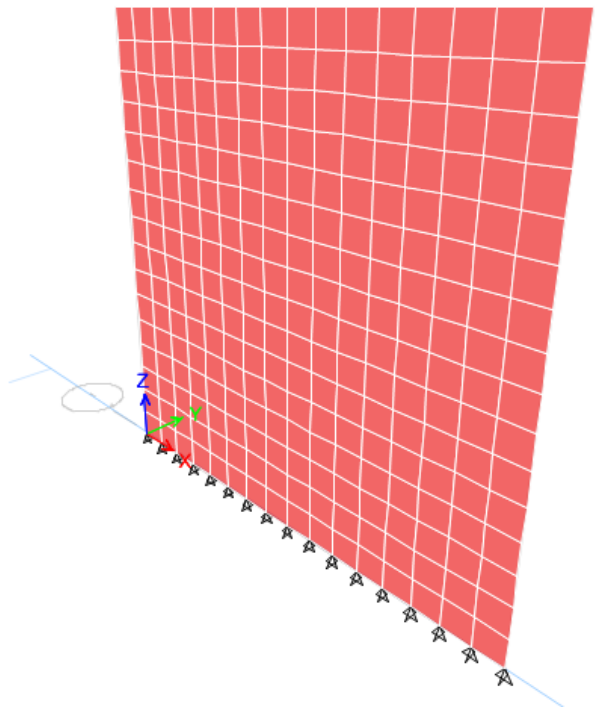


Figure G-2 Wall 2 (Example 2) Finite Element Mesh and Capacity Curve from FEM/I

- The incremental dynamic analysis (IDA) of the Example 1 and 2 walls was performed using IIIDAP Software (Lignos 2014) using the FEMA P695 method which considers 44 ground motions (22 pairs in two orthogonal directions). The median collapse intensities obtained from this analysis were used to calculate the Collapse Margin Ratio (CMR) per FEMA P695. A sample of the IDA results are shown in Figure G-3.

  Wall 1: CMR > 5.0 for a typical site class D in Los Angeles, CA.

  Wall 2: CMR = 3.9 for a typical site class D in Los Angeles, CA.

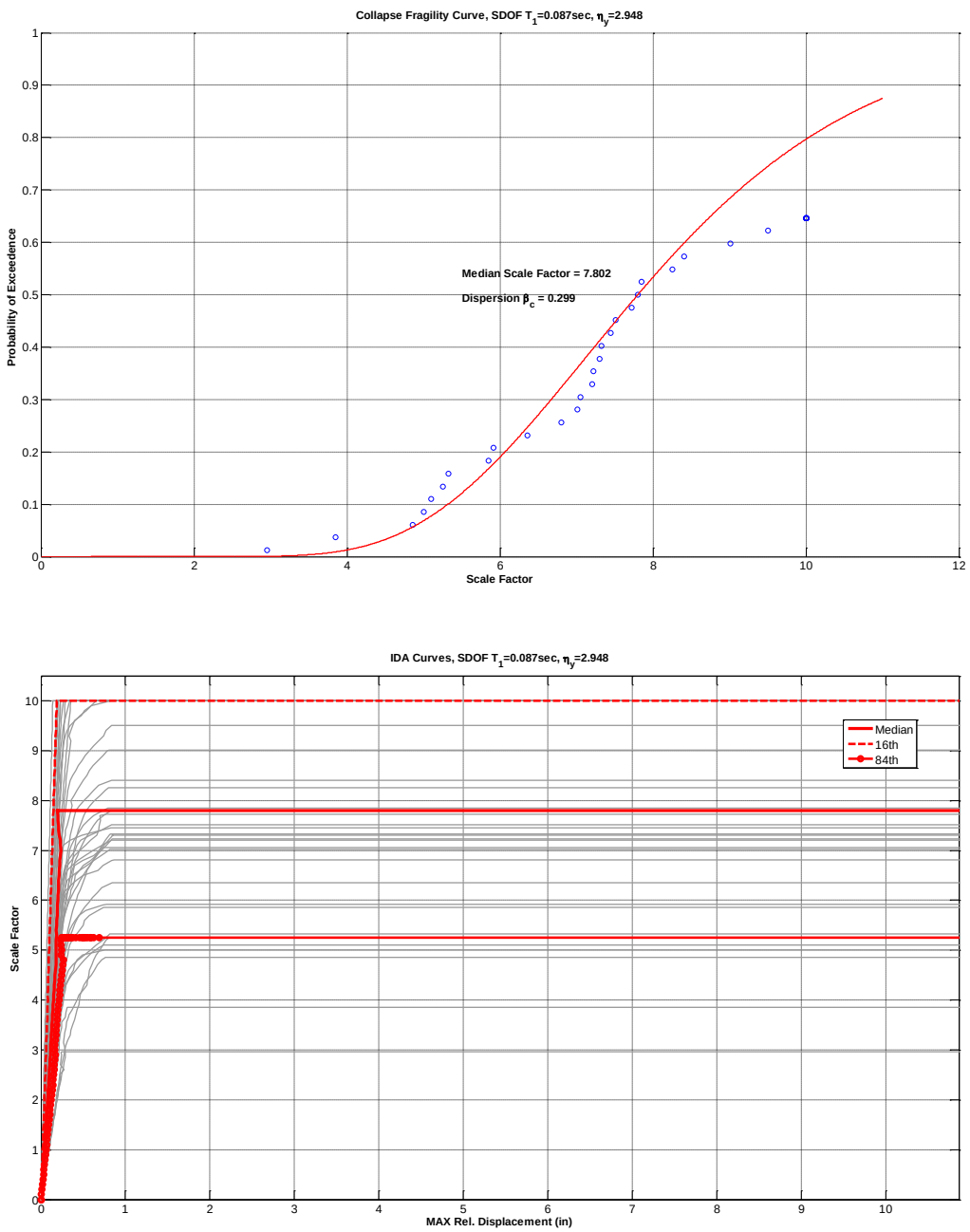


Figure G-3 Wall 2 (Example 2) IDA Results from IIIDAP

- Adjusted Collapse Margin Ratio (ACMR) was calculated by multiplying the CMRs by a spectral shape factor (SSF) obtained from FEMA P695 based on fundamental period and ductility.

  Wall 1: ACMR > 5.65

  Wall 2: ACMR = 4.76

- Collapse total uncertainty (β_{TOT}) was calculated per FEMA P695 as a function of the quality ratings associated with the design requirements, test data, nonlinear models, and record-to-record uncertainty.

$$\beta_{TOTAL} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}$$

  Where

  β_{TOTAL} = total system collapse uncertainty = 0.40 (based on the values below)

  β_{RTR}^2 = square of record-to-record collapse uncertainty = $(0.30)^2$ per FEMA P695 equation 7-2

  β_{DR}^2 = square of design requirement related collapse uncertainty = $(0.10)^2$ (Superior Confidence)

  β_{TD}^2 = square of test data-related collapse uncertainty = $(0.20)^2$ (Good Confidence)

  β_{MDL}^2 = square of modeling-related collapse uncertainty = $(0.15)^2$ (Superior-to-Good Accuracy and Robustness of Models)

- The acceptable values of ACMR for different collapse probabilities are specified in FEMA P695 based on total collapse uncertainty (β_{TOT}):

  ACMR_03% = 2.0 (extrapolated)

  ACMR_05% = 1.93

  ACMR_10% = 1.67

  ACMR_15% = 1.51

  ACMR_20% = 1.40

  ACMR_25% = 1.31

- The ACMR values calculated for both Wall 1 and Wall 2 are much larger than the acceptable values of ACMRs (listed above) for different probabilities of collapse and they meet the maximum probability of collapse criteria of ASCE 7-10 for structures in all types of Risk Categories I through IV, including essential facilities (Table G-1).

Quality Rating for Design Requirements

Completeness and Robustness	Confidence in Basis of Design Requirements	
	High	Medium
High. Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	(A) Superior $\beta_{DR} = 0.10$	(B) Good $\beta_{DR} = 0.20$
Medium. Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	(B) Good $\beta_{DR} = 0.20$	(C) Fair $\beta_{DR} = 0.35$

Completeness and Robustness Characteristics

- High. Design requirements are extensive, well-vetted and provide extensive safeguards against unanticipated failure modes. All important issues regarding system behavior have been addressed, resulting in a high reliability in the behavior of the system. Through mature construction practices, and tightly specified quality assurance requirements, there is a high likelihood that the design provisions will be well executed through fabrication, erection and final construction.
- Medium. Design requirements are reasonably extensive and provide reasonable safeguards against unanticipated failure modes, leaving some limited potential for the occurrence of such modes. Design requirements establish a suggested hierarchy of component yielding and failure. While most important behavioral issues have been addressed, some have not, which somewhat reduces the reliability of the system. Quality assurance requirements are specified but do not fully address all the important aspects of fabrication, erection and final construction.

Confidence in Design Requirements

- High. There is substantiating evidence (experimental data, history of use, similarity with other systems) that results in a high level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended.
- Medium. There is some substantiating evidence that results in a moderate level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended.

Quality Rating of Test Data

Completeness and Robustness	Confidence in Test Results	
	High	Medium
High. Material, component, connection, assembly, and system behavior well understood and accounted for. All, or nearly all, important testing issues addressed.	(A) Superior $\beta_{TD} = 0.10$	(B) Good $\beta_{TD} = 0.20$
Medium. Material, component, connection, assembly, and system behavior generally understood and accounted for. Most important testing issues addressed.	(B) Good $\beta_{TD} = 0.20$	(C) Fair $\beta_{TD} = 0.35$

Completeness and Robustness Characteristics

- **High.** All, or nearly all, important general testing issues are addressed comprehensively in the testing program and other supporting evidence. Experimental evidence is sufficient so that all, or nearly all, important behavior aspects at all levels (from material to system) are well understood, and the results can be used to quantify all important parameters that affect design requirements and analytical modeling.

- **Medium.** Most of the important general testing issues are addressed adequately in the testing program and other supporting evidence. Experimental evidence is sufficient so that all, or nearly all, important behavior aspects at all levels (from material to system) are generally understood, and the results can be used to quantify or deduce most of the important parameters that significantly affect design requirements and analytical modeling.

Confidence in Test Results

- **High.** Reliable experimental information is produced on all important parameters that affect design requirements and analytical modeling. Comparable tests from other testing programs have produced results that are fully compatible with those from the system-specific testing program. A sufficient number of tests are performed so that statistical variations in important parameters can be assessed. Test results are fully supported by basic principles of mechanics.
- **Medium.** Moderately reliable experimental information is produced on all important parameters that affect design requirements and analytical modeling. Comparable tests from other testing programs do not contradict, but do not fully corroborate, results from the system-specific testing program. A measure of uncertainty in important parameters can be estimated from the test results. Test results are supported by basic principles of mechanics.

Quality Rating of Index Archetype Models

Representation of Collapse Characteristics	Accuracy and Robustness of Models	
	High	Medium
High. Index models capture the full range of the archetype design space and structural behavioral effects that contribute to collapse.	(A) Superior $\beta_{MDL} = 0.10$	(B) Good $\beta_{MDL} = 0.20$
Medium. Index models are generally comprehensive and representative of the design space and behavioral effects that contribute to collapse.	(B) Good $\beta_{MDL} = 0.20$	(C) Fair $\beta_{MDL} = 0.35$

Representation of Collapse Characteristics

- **High.** The set of index archetype configurations and associated archetype models provides a complete and comprehensive representation of the full range of structural configurations, design parameters and behavioral characteristics that affect structural collapse. The index archetype models cover a comprehensive range of building heights, lateral system configurations, and design alternatives that are permitted by the design requirements. To the extent that 3-D component and system effects are significant, they are reflected in the index archetype models, as are other significant system effects such as diaphragm flexibility,
- **Medium.** The set of index archetype models provides a reasonably broad and complete representation of the design space. Where the complete design space is not fully represented in the set of models, there is reasonable confidence that the range of response captured by the models is indicative of the primary structural behavior characteristics that affect collapse.

Accuracy and Robustness of Models

- **High.** Nonlinear models directly simulate all predominate inelastic effects, from the onset of yielding through strength and stiffness degradation causing collapse. Models employ either concentrated hinges or distributed finite elements to provide spatial resolution appropriate for the proposed system. Computational solution algorithms are sufficiently robust to accurately track inelastic force redistribution, including cyclic loading and unloading, without convergence problems, up to the point of collapse.
- **Medium.** Nonlinear models capture most, but not all, nonlinear deterioration and response mechanisms leading to collapse. Models may not be sufficiently robust to track the full extent of deterioration, so that some component-based limit state checks are necessary to assess collapse.

APPENDIX H AUTHOR RESUMES

GARY C. HART, PHD, PE

Gary C. Hart is a Principal and Emeritus Board of Director of Weidlinger Associates® Inc. He received his BS in Civil Engineering from USC in 1965 and his MS and Ph.D. in Structural Engineering from Stanford University in 1966 and 1968, respectively. He was a tenured Professor in Structural Engineering at the University of California, Los Angeles from 1968 to 2001. His structural engineering career has been especially focused on building design and building performance estimation during severe earthquakes and winds. Dr. Hart has also served as a structural engineering expert witness for over 20 years and has testified many times and has serviced as a court appointed expert for the Supreme Court of California. Selected special recognition includes: Founding Editor, "International Journal on the Structural Design of Tall and Special Buildings," John Wiley & Sons; ASCE Ernest E. Howard Award for "definite contribution to the advancement of structural engineering"; ASCE Merit Award as Project Director and Structural Engineer of Record, "a national project that best illustrates superior civil engineering skills and represents a significant contribution to civil engineering progress and society"; and Founding President, Los Angeles Tall Buildings Structural Design Council. Selected notable building designs he worked on are the Rockwell building in Seal Beach (ASCE Building of the Year), Los Angeles City Hall, Wiltern Theater, Kerckhoff and Royce Halls (UCLA), several Kaiser Permanente Hospitals, and the Cathedral of Our Lady of Angels.

His service in legal disputes is approximately divided between Plaintiff and Defense positions and the legal counsel he has worked for have represented owners, insurance companies and design professionals. He has testified in Federal and State Court and for a range of project sizes, including one of the largest construction defect suits tried in the United States courts.

C. CAN SIMSIR, PHD, PE

Dr. Simsir has experience in earthquake damage investigation, hurricane and tornado damage investigation, failure investigation, construction defect investigation, assessment of seismic and wind loads on structures, seismic retrofit design for existing and heritage buildings, and structural design of buildings, underground terminals and rocket launch pads. Dr. Simsir's work includes both natural and man-made hazards. He has provided expert services in California, Hawaii, and New Zealand for post-earthquake site investigation and structural analysis, recommended repairs of numerous buildings, and testified in court as part of damage evaluations used in insurance dispute resolution. He has provided expert services for post-hurricane site investigation in Florida, Louisiana, Mississippi, Texas, and New Jersey and wind load assessment of numerous condominium buildings in Florida and New Jersey as part of damage evaluations used in insurance dispute resolution. He has provided expert services for construction defect investigation and recommended repairs of condominium and hotel towers in California, Hawaii, and Nevada as part of building evaluations used in insurance dispute resolution. He has been also involved in Weidlinger's structural engineering investigation and insurance dispute resolution of the 9/11 collapse of World Trade Center towers 1, 2, 5, and 7.

Dr. Simsir has conducted research in the field of unreinforced masonry structures. He is the developer of various dynamic stability evaluation methods for unreinforced masonry out-of-plane walls based on experimental, analytical, and observed building response during earthquakes. He has worked with wood-frame buildings to assess their seismic response using nonlinear static and dynamic analysis methods based on load-deformation characteristics of the shear walls and connecting nails. He is also the developer of various educational tools including low-cost accelerometers and moment-resistant frame structures for student laboratory projects on performance-based seismic design.

Dr. Simsir has authored or co-authored more than 30 publications including papers in refereed journals, conference proceedings, and technical reports. He has been an invited speaker or presenter at several national and international meetings and conferences. He has peer-reviewed articles for several scholarly journals and conference proceedings. His papers have been cited in a variety of scholarly journals and conference proceedings. A list of his publications is available upon request.