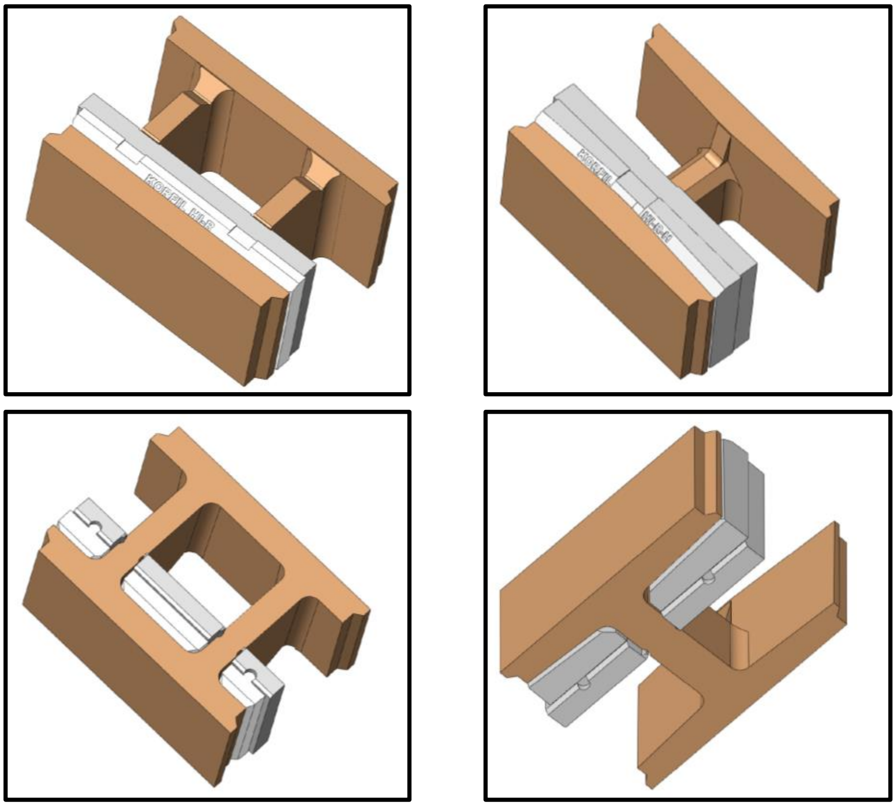
Supplemental Report: Reference Material for Shear-Dominated Walls Report

January 22, 2016 | Report 15-35a



For

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by

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FOREWORD

The reason for separating one report into two is based on my personal style of learning since the late 1960's. I was and continue to be a very slow reader and typically read good reports and journal papers two, three or more times. In each reading, I highlight and underline what I considered the key items that I wanted to remember and use in my UCLA classes, talks and future technical papers.

Nowadays, I type or scan the key points of my reading and therefore have an electronic file that can be sent through the Internet or, as with this Supplement Report, shared with readers. In a small but important way this approach provides credit to the authors of the excellent work that went into the quotations. Also I find it to be an important education / transfer technology aid.

This Supplemental Report provides the quotations that I and Dr. Simsir selected to share in the context of this report.

We have some more work to do on this Supplemental Report over the next two weeks so please pass on to us any questions that you wish us to consider. With your help we can advance the application of Performance Based Design and reward innovation.

Gary C. Hart
Principal Emeritus, Thornton Tomasetti
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Table of Contents

APPENDIX 1 FEMA 306: QUOTATIONS

RM1B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System: **Reinforced Masonry**
		Component Type: **Stronger Pier**
		Behavior Mode: **Flexure / Shear**
		Applicable Materials: **Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

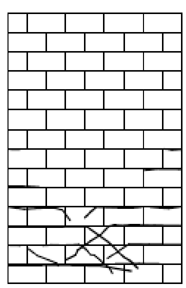
Damage in an RM1 component with a flexural /shear response is typically localized to the base of the wall, within the plastic hinge region. Both horizontal and diagonal cracks will be present, with diagonal cracks predominant. Diagonal cracks may appear to be independent from horizontal, flexural cracks, and may propagate across the major diagonal dimensions. At heavy damage levels, shear deformations are likely to be localized to one or two diagonal cracks of large width. If a permanent horizontal offset is visible, the behavior mode may be Flexure/Sliding Shear

By analysis:

Analysis of a wall with a Flexure / Shear behavior mode may be difficult, with no clear distinction between the controlling mechanism of flexure (deformation-controlled) or shear (force-controlled). Calculated capacities should be in the same range. Wall axial loads may be moderate-to-high.

Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response
- Evaluation of plastic hinge length
- Identifying flexural versus shear cracks.
- Crack evaluation.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16", <u>and</u> • No significant spalling *Typical Appearance:* 	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/8", <u>and</u> • No significant spalling or vertical cracking *Typical Appearance:* Similar to insignificant damage except cracks are wider with more extensive cracking.	• Inject cracks $\lambda_K^* = 0.9$ $\lambda_Q^* = 1.0$ $\lambda_D^* = 1.0$

124 Basic Procedures Manual FEMA 306

RM2B	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Flexure / Shear**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

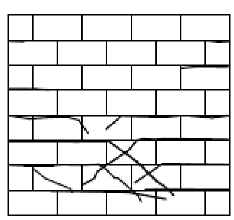
Damage in an RM2 component with a flexural/shear response may be localized to the first story, or it may be evident at a number of levels in story-height piers. Both horizontal and diagonal cracks may be present, with diagonal cracks predominant. Diagonal cracks may appear to be independent from horizontal flexural cracks, and propagate across the major diagonal dimensions. When severely damaged, shear deformations will be localized to one or two diagonal cracks of large width. If diagonal cracks are uniformly distributed and of small width, the behavior mode may be ductile flexure. If a permanent horizontal offset is visible, the behavior mode may include Flexure/Sliding Shear.

By analysis:

Analysis of a wall with a Flexure / Shear behavior mode may not indicate a clear distinction between the controlling mechanism of flexure (deformation controlled) or shear (force controlled). Calculated capacities should be in the same range. Wall axial loads may be moderate to high.

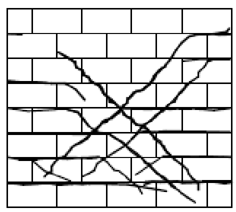
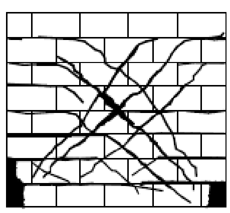
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response.
- Identifying flexural versus shear cracks.
- Crack width discussion.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/16." • No significant spalling. *Typical Appearance:*  May appear similar to flexure following small displacement cycles. Diagonal cracks often propagate from horizontal cracks.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM2B

Severity	**Description of Damage**	**Performance Restoration Measures**
Slight $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* • No crack widths exceed 1/8”. • No significant spalling or vertical cracking. *Typical Appearance:* Similar to insignificant damage, except cracks are wider and cracking is more extensive.	• Inject cracks. $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Crack widths do not exceed 3/16”. • Moderate spalling of masonry unit faceshells or vertical cracking at toe regions. • No buckled or fractured reinforcement. • No significant residual displacement. *Typical Appearance:* 	• Remove and patch spalled masonry and loose concrete. Inject cracks. • Consider horizontal fiber composite overlay. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Extreme	*Criteria:* • Reinforcement has fractured *Typical Indications* • Wide flexural cracking typically > ¼” concentrated in a single crack. • Wide diagonal cracking, typically concentrated in one or two cracks • Extensive crushing or spalling at wall toes, visible delamination of faceshells from grout *Typical Appearance* 	• Replacement or extensive enhancement required.

RM2G	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Reinforced Masonry**
		Component Type:	**Weaker Pier**
		Behavior Mode:	**Preemptive Shear**
		Applicable Materials:	**Fully grouted hollow concrete or clay units**

How to distinguish behavior mode:

By observation:

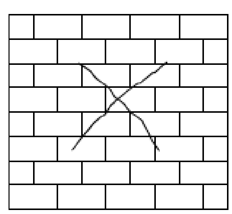
At low levels of damage, wall may appear similar to RM2B. Diagonal cracks may be visible before flexural cracks. Damage occurs quickly in the form of one or two dominant diagonal cracks. Subsequent cycles may cause crushing or face shell debonding at the center of the wall and/or at the wall toes.

By analysis:

Calculated shear load capacity, including both masonry and steel components, will be less than or equal to shear associated with flexural load capacity

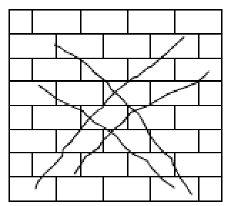
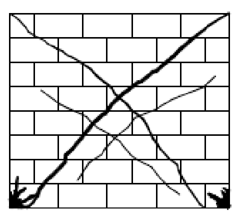
Refer to Evaluation Procedures for:

- Evaluation of flexural response.
- Evaluation of shear response.
- Evaluation of crack patterns.
- Crack evaluation.

Severity	Description of Damage		Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:*	• No diagonal cracks. • Flexural crack <1/16". • No significant spalling.	Not necessary for restoration of structural performance. (Cosmetic measures may be necessary for restoration of nonstructural characteristics.)
	Typical Appearance:	No visible damage.	
Slight $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:*	• No crack widths exceed 1/16". • No significant spalling or vertical cracking.	• Inject cracks. $\lambda_K{}^* = 0.9$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
	Typical Appearance:	Similar to insignificant damage, except that small diagonal cracks may be present. 	

COMPONENT DAMAGE
CLASSIFICATION GUIDE **continued**

RM2G

Severity	Description of Damage	Performance Restoration Measures
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* • Crack widths do not exceed 1/16". • No spalling of masonry unit faceshells or vertical cracking at toe regions. *Typical Appearance:* May be several diagonal cracks, typically with one dominant crack. 	• Inject cracks. • Consider horizontally oriented fiber composite overlay. $\lambda_K{}^* = 0.8$ $\lambda_Q{}^* = 1.0$ $\lambda_D{}^* = 1.0$
Heavy $\lambda_K = 0.3$ $\lambda_Q = 0.4$ $\lambda_D = 0.5$ See FEMA 307 for calculation of λ_Q	*Criteria:* • Single dominant crack, may be > 3/8". *Typical Appearance:* 	• Inject cracks. • Provide horizontally oriented fiber composite overlay. • Consider replacement.
Extreme	*Criteria:* • Reinforcement has fractured. *Typical Indications* • Wide diagonal cracking, typically concentrated in one or two cracks. • Crushing or spalling at center of wall or at wall toes.	• Replacement or enhancement required.

APPENDIX 2 FEMA P695/ATC 63 and FEMA P795/ATC 63-1 REPORTS: QUOTATIONS

J.1 ATC-63 Sources of Uncertainty

3.4 Quality Rating for Design Requirements

"Quantitative values of design requirements-related collapse uncertainty are: (A) Superior, β_{DR} = 0.10; (B) Good, β_{DR} = 0.20; (C) Fair, β_{DR} = 0.35; and (D) Poor, β_{DR} = 0.50." (ATC-63/FEMA P695, pg. 3-8)

Table 3-1 Quality Rating of Design Requirements (ATC-63/FEMA P695, pg. 3-8)

Completeness and Robustness	Confidence in Basis of Design Requirements		
	High	Medium	Low
High. Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	(A) Superior β_{DR} = 0.10	(B) Good β_{DR} = 0.20	(C) Fair β_{DR} = 0.35
Medium. Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	(B) Good β_{DR} = 0.20	(C) Fair β_{DR} = 0.35	(D) Poor β_{DR} = 0.50
Low. Questionable safeguards against unanticipated failure modes. Many important design and quality assurance issues are not addressed.	(C) Fair β_{DR} = 0.35	(D) Poor β_{DR} = 0.50	--

"The lowest rating of (D) Poor applies to design requirements that have minimal safeguards against unanticipated failure modes, do not ensure a hierarchy of yielding and failure, and would generally be associated with systems that exhibit behavior that is difficult to predict." (ATC-63/FEMA P695, pg. 3-9)

3.4.2 Confidence in Design Requirements

"Confidence in the basis of the design requirements refers to the degree to which the prescribed material properties, strength criteria, stiffness parameters, and design equations are representative of actual behavior and will achieve the intended result. Confidence is rated from high to low, as follows:

- **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.

- **Low.** There is little substantiating evidence (little experimental data, no history of use, no similarity with other systems) that results in a low level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended." (ATC-63/FEMA P695, pg. 3-10)

3.5 Data from Experimental Investigation

"Analytical modeling alone is not adequate for predicting nonlinear seismic response with confidence, particularly for structural systems that have not been subjected to past earthquakes." (ATC-63/FEMA P695, pg. 3-10)

"A comprehensive experimental investigation program is necessary to establish material properties, develop design criteria, calibrate and validate component models, confirm behavior, and calibrate analyses for a proposed seismic-force-resisting system." (ATC-63/FEMA P695, pg. 3-10)

"...limitations on available experimental data will affect the uncertainty and reliability of the collapse assessment of a proposed system, and will factor directly into the performance evaluation process. The scope of an experimental investigation program should be developed in consultation with the peer review panel." (ATC-63/FEMA P695, pg. 3-11)

3.6 Quality Rating of Test Data

"Quality of test data is related to uncertainty, which factors into the performance evaluation for a proposed seismic-force-resisting system. The quality of test data obtained from an experimental investigation program is rated in accordance with the requirements of this section, and approved by the peer review panel. (ATC-63/FEMA P695, pg. 3-19)

"Test data are rated between (A) Superior and (D) Poor, as shown in Table 3-2. This rating depends not only on the quality of the testing program, but on how well the tests address key parameters and behavioral issues. The selection of a quality rating for test data considers the completeness and robustness of the overall testing program, and confidence in the test results. Quantitative values of test data-related collapse uncertainty are: (A) Superior, β_{TD} = 0.10; (B) Good, β_{TD} = 0.20; (C) Fair, β_{TD} = 0.35; and (D) Poor, β_{TD} = 0.50. Use of these values is described in Section 7.3." (ATC-63/FEMA P695, pg. 3-19)

Table 3-2 Quality Rating of Test Data from an Experimental Investigation Program
(ATC-63/FEMA P695, pg. 3-20)

Completeness and Robustness	Confidence in Test Results		
	High	Medium	Low
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 $\beta_{TD} = 0.20$	(C) Fair $\beta_{TD} = 0.35$
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 $\beta_{TD} = 0.35$	(D) Poor $\beta_{TD} = 0.50$
Low. Material, component, connection, assembly and system behavior fairly understood and accounted for. Several important testing issues not addressed.	(C) Fair $\beta_{TD} = 0.35$	(D) Poor $\beta_{TD} = 0.50$	--

3.6.1 Completeness and Robustness Characteristics

"Completeness and robustness characteristics are related to: (1) the degree to which relevant testing issues have been considered in the development of the testing program; and (2) the extent to which the testing program and other documented experimental evidence quantify the necessary material, component, connection, assembly, and system properties and important behavior and failure modes. Completeness and robustness characteristics are rated from high to low, as follows:

- **High.** All, or nearly all, important general testing issues of Section 3.5.2 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 of Section 3.5.2 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.

- **Low.** Several important general testing issues of Section 3.5.2 are not addressed adequately in the testing program and other supporting evidence. Experimental evidence is sufficient so that the most important behavior aspects at all levels (from material to system) are fairly well understood, but the results are not adequate to quantify or deduce, with high confidence, many of the important parameters that significantly affect design requirements and analytical modeling." (ATC-63/FEMA P695, pgs. 3-20 & 3-21)

3.6.2 Confidence in Test Results

"Confidence in test results is related to the reliability and repeatability of the results obtained from the testing program, and corroboration with available results from other relevant testing programs. It includes consideration as to whether or not experimental results consistently record performance to failure for all modes of behavior (limited ductility to large ductility), and if sufficient information is provided to assess uncertainties in the design requirements (e.g., ▯ factors) and analytical models. Confidence in test results is rated from high to low, as follows:

- **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.

- **Low.** Experimental information produced on many of the important parameters that affect design requirements and analytical modeling is of limited reliability. Comparable tests from other testing programs do not support the results from the system-specific testing program. Insufficient data exists to assess uncertainty in many important parameters. Basic principles of mechanics do not support some of the results of the testing program." (ATC-63/FEMA P695, pg. 3-21)

5.6 Characterization of Modeling Uncertainties

"In this Methodology, nonlinear analysis is used to determine the median ground motion intensity associated with collapse of a proposed seismic-force- resisting system. Index archetype models should, therefore, represent the median response of structural components that constitute the proposed system. Variability in collapse response, due to ground motion variability, modeling, and other uncertainties, is factored into the performance evaluation process in Chapter 7. When a model calibrated to median properties is used, nonlinear dynamic analysis under multiple ground motions is intended to provide a median estimate of the collapse capacity of an index archetype." (ATC-63/FEMA P695, pg. 5-22)

5.7 Quality Rating of Index Archetype Models

"Quality of index archetype models is related to uncertainty, which factors into the performance evaluation for a proposed seismic-force-resisting system. The quality of index archetype models is rated in accordance with the requirements of this section, and approved by the peer review panel." (ATC-63/FEMA P695, pg. 5-23)

"Index archetype models are rated between (A) Superior and (D) Poor, as shown in Table 5-3. This rating is a combined assessment of: (1) how well index archetype models represent the range of structural collapse characteristics and associated design parameters of the archetype design space; and (2) how well the analysis models capture structural collapse behavior through both direct simulation and non-simulated limit state checks. The quantitative values of modeling-related collapse uncertainty are: (A) Superior, β_{MDL} = 0.10; (B) Good, β_{MDL} = 0.20; (C) Fair, β_{MDL} = 0.35; and (D) Poor, β_{MDL} = 0.50. Use of these values is described in Section 7.3." (ATC-63/FEMA P695, pg. 5-23)

Table 5-3 Quality Rating of Index Archetype Models (ATC-63/FEMA P695, pg. 5-23)

Representation of Collapse Characteristics	Accuracy and Robustness of Models		
	High	Medium	Low
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 $\beta_{MDL} = 0.20$	(C) Fair $\beta_{MDL} = 0.35$
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 $\beta_{MDL} = 0.35$	(D) Poor $\beta_{MDL} = 0.50$
Low. Significant aspects of the design space and/or collapse behavior are not captured in the index models.	(C) Fair $\beta_{MDL} = 0.35$	(D) Poor $\beta_{MDL} = 0.50$	--

"The highest rating of (A) Superior applies to instances in which the index archetype models represent the complete range of structural configuration and collapse behavior, there is a high confidence in the ability of established models to simulate behavior, and the nonlinear model is of high-fidelity. The combination of low quality representation of collapse characteristics along with low quality modeling in terms of accuracy and robustness is not permitted." (ATC-63/FEMA P695, pg. 5-23)

"An adaptation of the Methodology to assess building-specific collapse performance of an individual building is presented in Appendix F. Differences in assigning quality ratings for an analytical model of an individual building are discussed there." (ATC-63/FEMA P695, pgs. 5-23 & 5-24)

5.7.1 Representation of Collapse Characteristics

"Representation of collapse characteristics refers to how completely and comprehensively the index archetype models capture the full range of design parameters and associated structural collapse behavior that is envisioned within the archetype design space. The quality of the representation is characterized as follows:

- **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.

- **Low.** The set of index archetype models does not capture the full range of structural configurations and collapse behavior for the system due to the combined effects of a loosely defined design space and a less than complete set of index archetype configurations. Loosely defined limits on system configurations and design parameters present a challenge in that the number of possible alternative configurations and structural design parameters are so large as to preclude systematic interrogation with a manageable number of index archetype configurations. Seismic-force-resisting systems permitted in low Seismic Design Categories that have limited requirements on design (e.g., steel ordinary moment frame systems) may fall into this category. Even for well controlled design criteria, however, representation of collapse characteristics may be low if the number and variety of index archetype configurations are not insufficient to capture the possible range in collapse behavior." (ATC-63/FEMA P695, pg. 5-24)

5.7.2 Accuracy and Robustness of Models

"Accuracy and robustness is related to the degree to which nonlinear behaviors are directly simulated in the model, or otherwise accounted for in the assessment. Use of non-simulated collapse limit state checks will lower the accuracy and robustness of a nonlinear model. If conservatively applied, however, non-simulated collapse checks should not necessarily lower the overall quality rating of the assessment procedure. Model accuracy and robustness are characterized as follows:

- **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.

- **Low.** Nonlinear models capture the onset of yielding and subsequent strain hardening, but do not simulate degrading response. Onset of degradation is primarily evaluated using non-simulated component limit state checks. Overall uncertainty in response quantities is increased due to inability to capture the effects of deterioration and redistribution." (ATC-63/FEMA P695, pg. 5-25)

"**Record-to-Record Uncertainty (*RTR*).** ...Values of record-to-record variability, β_{RTR}, ranging from 0.35 to 0.45 are fairly consistent among various building types (Haselton, 2006; Ibarra and Krawinkler, 2005a and 2005b; Zareian et al., 2006; Zareian, 2006)." (ATC-63/FEMA P695, pg. 7-7)

"Studies in Appendix A also found that record-to-record variability can be significantly less than β_{RTR} = 0.40 for systems that have little, or no, period elongation (e.g., systems with very limited ductility and certain base-isolated systems). For these systems, values of record-to-record variability can be reduced as follows:

$$\beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.40 \qquad (7\text{-}2)$$

where β_{RTR} must be greater than or equal to 0.20." (ATC-63/FEMA P695, pg. 7-7)

APPENDIX 3 NIST GCR 14-917-31 SEISMIC DESIGN OF SPECIAL REINFORCED MASONRY WALLS

NIST GCR 14-917-31

NEHRP Seismic Design Technical Brief No. 9

Seismic Design of Special Reinforced Masonry Shear Walls

A Guide for Practicing Engineers

Gregory R. Kingsley
P. Benson Shing
Thomas Gangel

National Institute of Standards and Technology
U.S. Department of Commerce

NIST GCR 14-917-31

Seismic Design of Special Reinforced Masonry Shear Walls

A Guide for Practicing Engineers

Prepared for
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National Institute of Standards and Technology
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U.S. Department of Commerce
Penny Pritzker, Secretary

National Institute of Standards and Technology
Willie E. May, Acting Under Secretary of Commerce for Standards and Technology and Acting Director

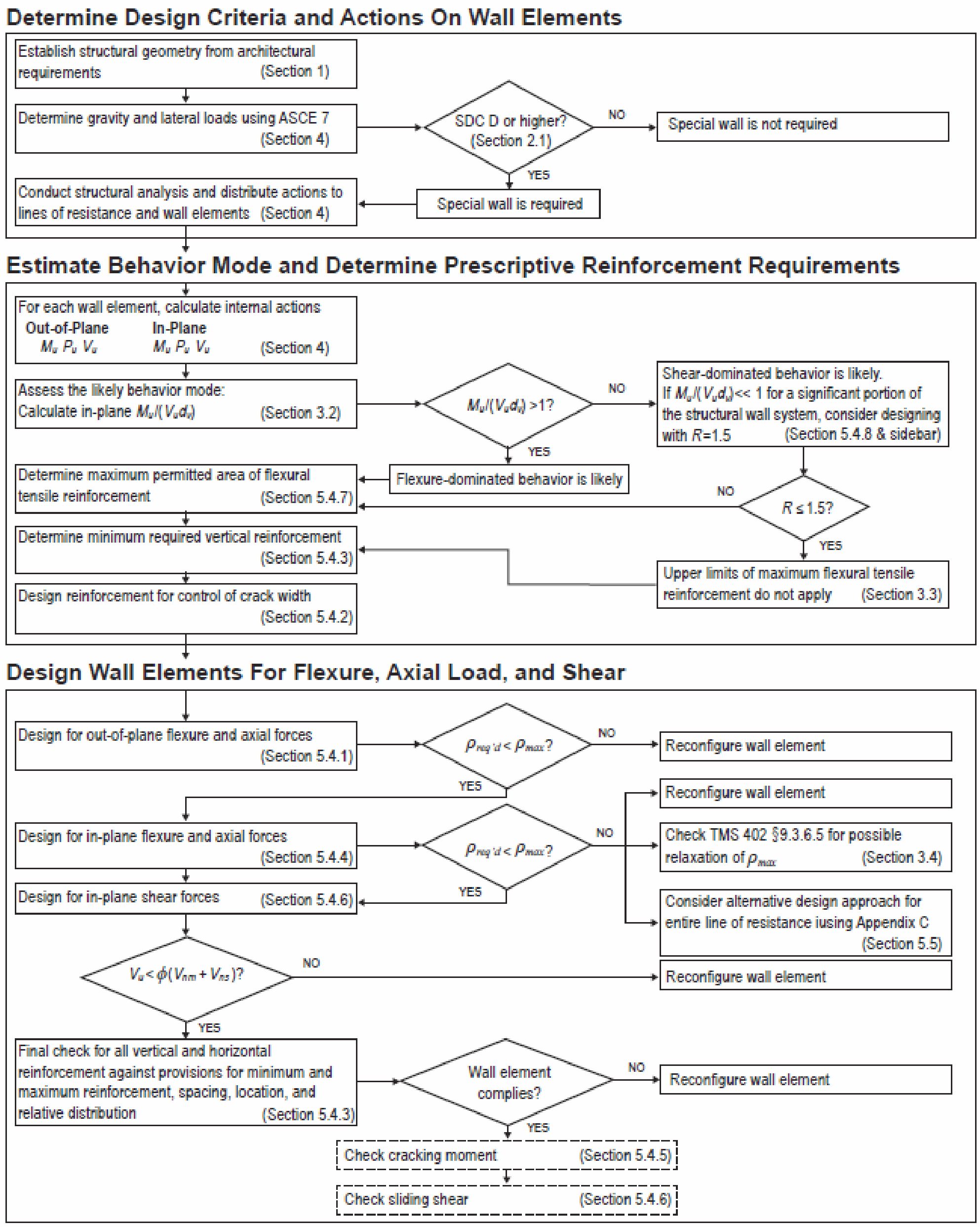


Figure 1-1. Flow chart of steps in the design of special reinforced masonry shear walls. Numbers in parenthesis cross-reference the sections in this Guide.

2. The Use of Reinforced Masonry Structural Walls in Buildings

2.1 Use of Special Reinforced Masonry Shear Walls

“Special reinforced masonry shear walls (“special walls”) are required to meet the most restrictive material and prescriptive detailing requirements. Accordingly, they are permitted by ASCE 7 to be used in any SDC per the judgment of the structural designer. Special walls are required to be used for reinforced masonry walls in SDC D, E, or F.” (NIST GCR 14-917-31, pg. 4)

“Special walls are assigned the highest response modification factor, *R*, of any of the masonry shear wall types.” (NIST GCR 14-917-31, pg. 4)

“… special reinforced masonry shear walls are assigned an *R* factor of 5; for special reinforced masonry wall building frame systems, *R* = 5.5 …” (NIST GCR 14-917-31, pg. 4)

“… Inherent in the use of an *R* factor of 5 or greater is the presumption of ductile behavior, associated with the development of plastic hinges with stable inelastic rotation capacity. Stable plastic hinges are characterized by the development of strains well past yield in the flexural reinforcement before the occurrence of flexural strength degradation or shear failure occurs in the wall.” (NIST GCR 14-917-31, pg. 4)

“… given the wide variety of masonry wall types and configurations and the lack of control of the structural designer over these configurations in many cases, the designer should not assume that following the prescriptive requirements alone will necessarily ensure ductile, flexure-dominated behavior.” (NIST GCR 14-917-31, pg. 4)

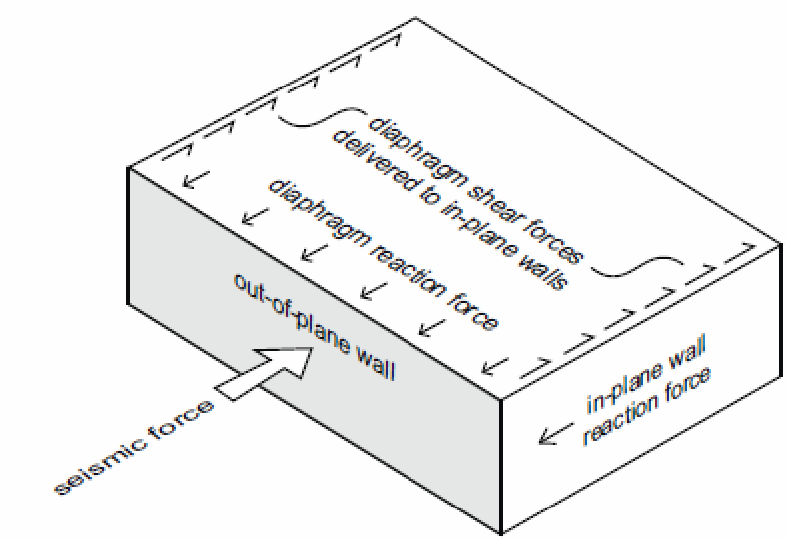


Figure 2-1. Typical load path through a masonry building.

(NIST GCR 14-917-31, pg. 4)

“Typical wall configurations are shown in elevation in Figure 2-3. <u>Squat wall elements</u> like those in Figures 2-3(a) and 2-3(b) with aspect ratios (height /plan length) of one or less are quite common, and they are often much stronger than required.” (NIST GCR 14-917-31, pg. 5) [Underline by Hart]

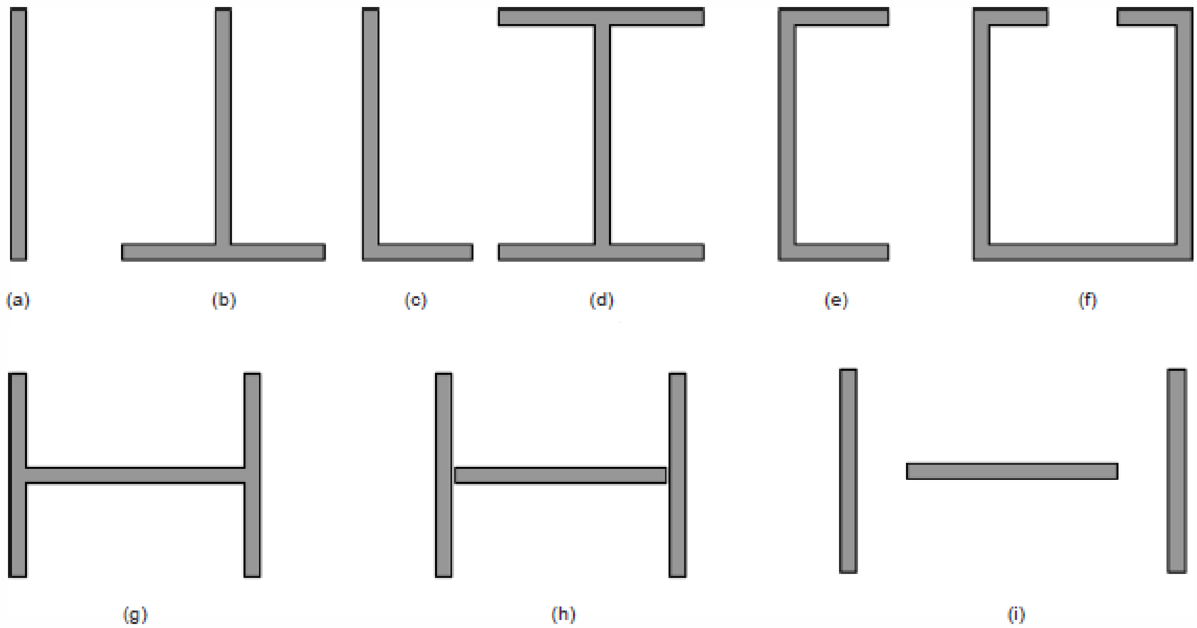
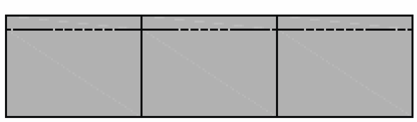


Figure 2-2. Plan configurations of walls: (a) typical shear wall, (b) T-shaped flanged wall, (c) L-shaped flanged wall, (d) I-shaped flanged wall, (e) C-shaped flanged wall, (f) box-section or core wall with an opening, (g) I-shaped wall with full continuity between web and flanges, (h) I-shaped wall with disconnected web and flanges and stiff coupling from floors, (i) I-shaped wall with flexible coupling from floors.

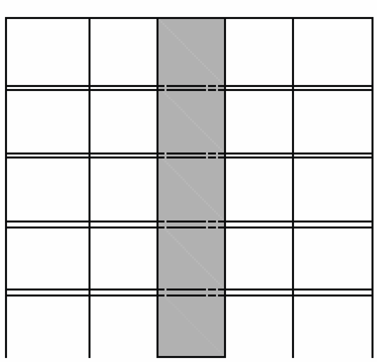
(NIST GCR 14-917-31, pg. 5)

"Tall cantilever walls or cores (Figure 2-3(e)) are the configuration most likely to display the flexure-dominated behavior that meets the intent of the code for special walls." (NIST GCR 14-917-31, pg. 6)



(a) Squat, shear-dominated wall, showing control joints

(NIST GCR 14-917-31, pg. 6)



(e) Cantilever wall

(NIST GCR 14-917-31, pg. 6)

3. Design Principles for Special Masonry Shear Walls

3.1 Allowable Stress Design, Strength Design, and Limit Design

"… In this Guide, the emphasis is on SD because TMS 402 addresses ductility requirements relevant to special walls more explicitly for SD than ASD." (NIST GCR 14-917-31, pg. 8)

"The 2013 edition of TMS 402 also includes a new Appendix C on Limit Design …" (NIST GCR 14-917-31, pg. 8)

"… Limit Design allows the structural designer to explicitly take into account the anticipated plastic mechanism of the wall system, to control the aspect ratios and detailing of wall elements to achieve the best behavior possible, and to detail the elements in accordance with the resulting flexure- or shear-dominated behavior." (NIST GCR 14-917-31, pg. 8)

3.2 Flexure-Dominated versus Shear-Dominated Walls

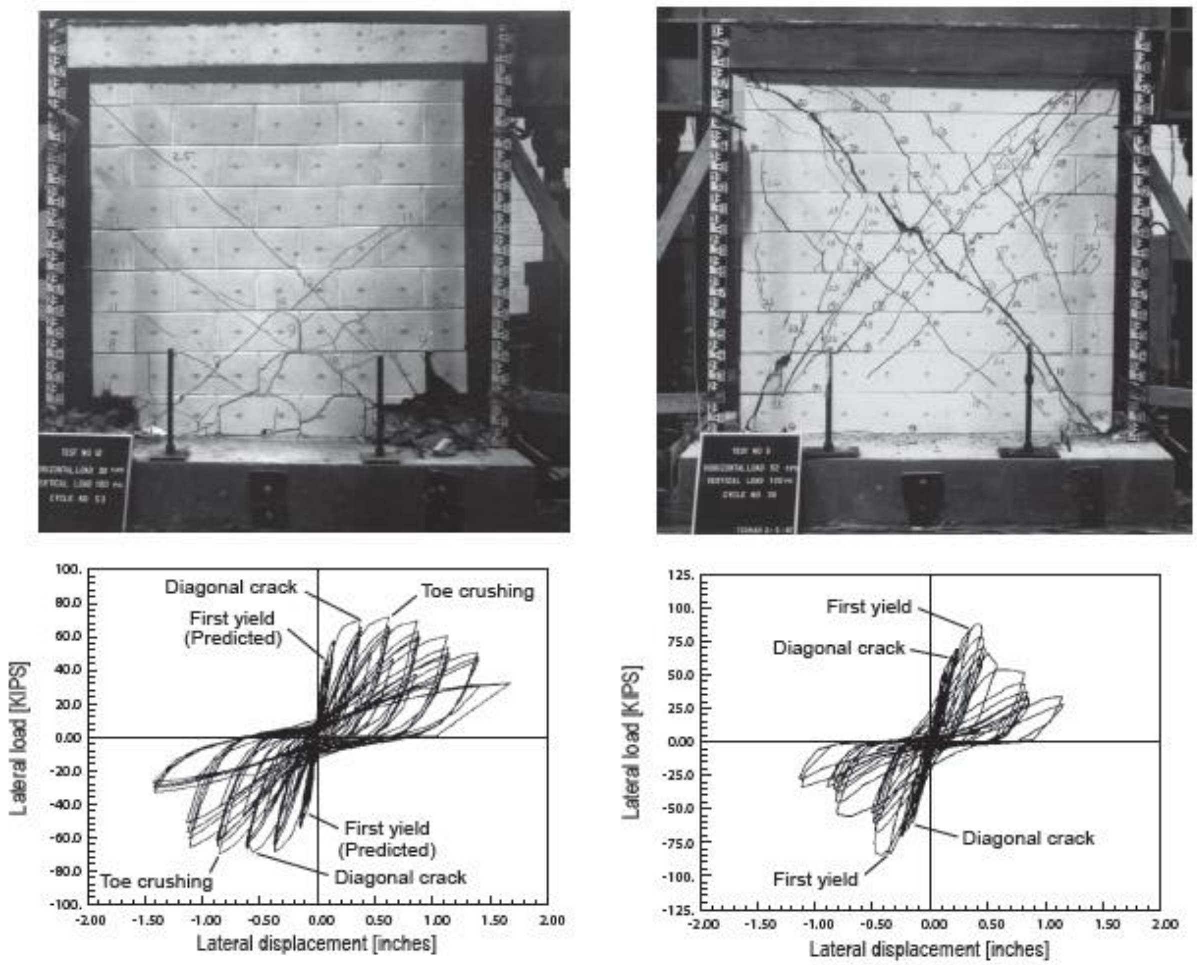
"A reinforced masonry wall system is composed of wall segments, each of which can be categorized as either flexure-dominated or shear-dominated. A <u>flexure-dominated wall</u> segment is one whose inelastic response is dominated by deformations resulting from the tensile yielding of flexural reinforcement. A <u>shear-dominated segment</u> is one whose inelastic response is dominated by diagonal shear (tension) cracks." (NIST GCR 14-917-31, pg. 8) [Underline by Hart]

"… Shear-dominated elements are generally brittle, with failure characterized by diagonal shear cracks." (NIST GCR 14-917-31, pg. 9)

"… when a special shear wall has a <u>shear-span-to-depth ratio less than one or a high axial load</u>, the same combination of prescriptive requirements may still result in a wall that is shear-dominated and brittle. This is often the case for low-rise masonry buildings, which constitute most masonry construction in the United States." (NIST GCR 14-917-31, pg. 9) [Underline by Hart]

"… $V_{flexure}$ is the shear demand associated with the <u>expected flexural capacity, which is 1.25M_n</u> divided by the wall height, with M_n being the nominal moment capacity, and V_{shear} is the <u>nominal shear strength V_n</u> calculated according to TMS 402." (NIST GCR 14-917-31, pg. 9) [Underline by Hart]

"… shear-dominated behavior becomes more likely as the amount of vertical (longitudinal) reinforcement increases, the amount of transverse reinforcement decreases, the wall length increases, or the axial compression force increases." (NIST GCR 14-917-31, pg. 9)



(a) Flexure-dominated wall (b) Shear-dominated wall

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

(NIST GCR 14-917-31, pg. 9)

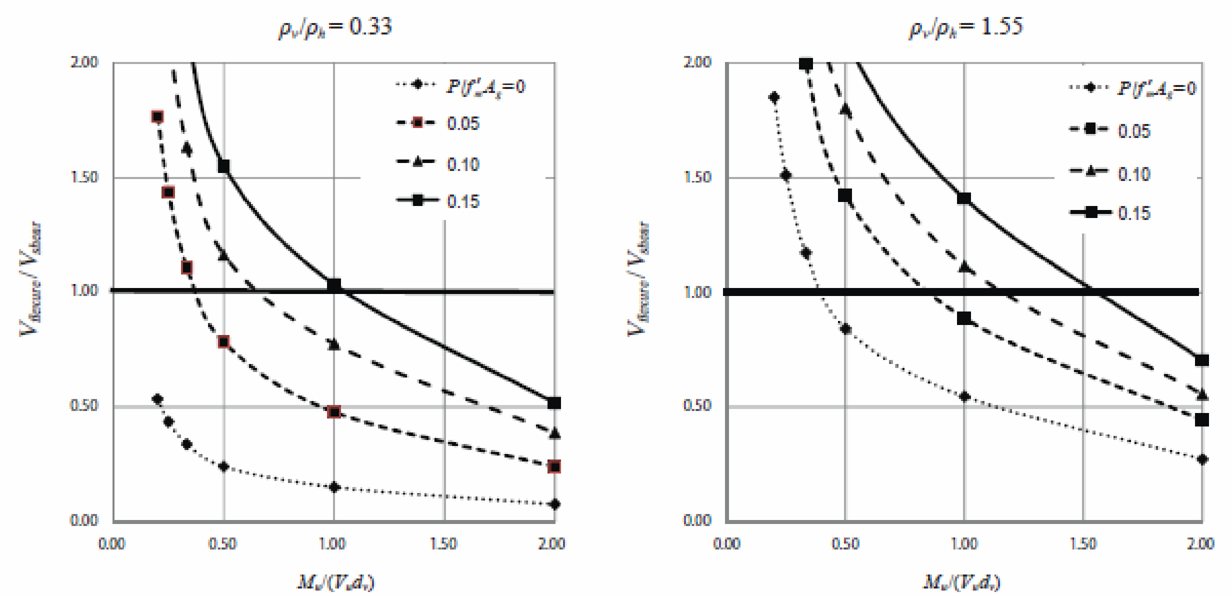


Figure 3-3. The effect of aspect ratio and axial load on the expected behavior of shear walls with two different ratios of vertical to horizontal reinforcement. Data points that fall above the line $V_{flexure}/V_{shear}$ = 1.0 represent walls most likely to have shear-dominated behavior.

(NIST GCR 14-917-31, pg. 10)

"Figure 3-3 is based on a simple cantilever wall loaded at the top. In a real structure, numerous effects such as higher-mode effects or axial forces and moments induced by coupling elements can amplify the shear that can be developed, corresponding to the moment capacity of the wall beyond that represented here." (NIST GCR 14-917-31, pg. 11) [Underline by Hart]

"To protect a special wall against shear failure caused by possible flexural overstrength, TMS 402 §7.3.2.6.1.1 requires that the design shear strength, ϕV_n , exceed the shear corresponding to the development of the nominal moment capacity by a factor of at least 1.25. The code states that the nominal shear strength, V_n, need not exceed 2.5 times the factored shear demand V_u…" (NIST GCR 14-917-31, pg. 11) [Underline by Hart]

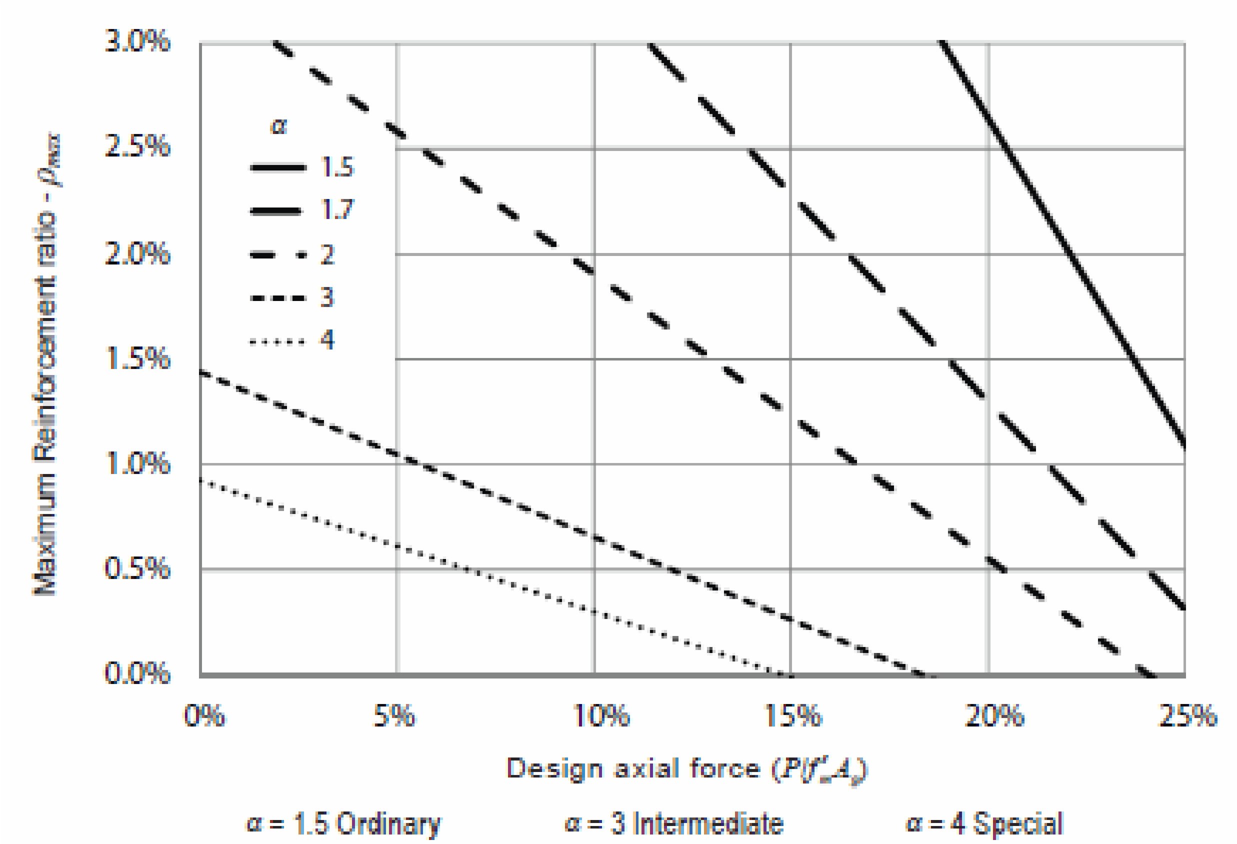


Figure 3-5. Maximum reinforcement ratios ρ_{max} using SD for in-plane walls with distributed reinforcement and varying levels of axial load, illustrating the effect of varying values for α. Only α values of 1.5, 3, and 4 are relevant to the code.

(NIST GCR 14-917-31, pg. 11)

"For squat walls with $M_u/(V_u d_v) < 1.0$, TMS 402 §9.3.3.5.4 allows the designer to design the wall for amplified forces–effectively, the forces associated with elastic response–in which case there is no upper limit to the maximum flexural tensile reinforcement." (NIST GCR 14-917-31, pg. 12)

"… the ASD provisions in TMS 402 §8.3.4.4 have no maximum reinforcement limitations for shear walls with $M/(Vd_v) \leq 1.0$ and an axial load ratio $P/f'_m A_n \leq 0.05$." (NIST GCR 14-917-31, pg. 12)

APPENDIX 4 PROFESSIONAL PAPERS: QUOTATIONS

SHEAR DESIGN OF STRUCTURAL WALL
(ACI 318-11 from Wallace, et al paper)

<u>Nominal Shear Strength</u> $\phi V_n \geq V_u$

$$V_n = A_{cv}\left[\alpha_c \lambda \sqrt{f_c^{'}} + \rho_t f_y\right]$$

$f_c^{'}$, f_y = Specified Compressive Strength and Yield Strength

α_c = Coefficient defining the relative contribution of concrete strength to nominal wall shear strength

ρ_t = Ratio of area of transverse reinforcement to gross concrete area perpendicular to that reinforcement

<u>Now consider</u> α_c

α_c = 3 for $\left({}^{hw}\!/_{\ell w}\right) \leq 1.5$

$\left({}^{hw}\!/_{\ell w}\right) \leq 1.5$ is $\ell w \geq \left({}^{hw}\!/_{1.5}\right) = \left(\dfrac{2}{3}\right) hw$

Therefore, if the wall length is equal to or greater than (2/3) the height of the wall, then α_c = 3.

A limit of the wall being longer than the height, i.e. $\ell w >$ hw is clearly greater than (2/3), i.e. it is 1 so the wall is classified as an α_c = 3 wall.

INTERACTIVE INTERFACE FOR INCREMENTAL DYNAMIC ANALYSIS PROCEDURE (IIIDAP) USING DETERIORATING SINGLE DEGREE OF FREEDOM SYSTEMS (Quotations)

By

Dimitrios G. Lignos, Ph.D.

March, 2010

INCREMENTAL DYNAMIC ANALYSIS PROCEDURE

"Interactive Interface for Incremental Dynamic Analysis Procedure (IIIDAP) software is a generic single degree of freedom analysis (SDOF) software for seismic evaluation of deteriorating and non-deteriorating SDOF systems. The software uses deteriorating hysteretic models that can adequately capture all the important deterioration modes of a component and is able to simulate collapse of SDOF systems under seismic loading." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 5)

"The modified Ibarra – Krawinkler deterioration model is defined by a backbone curve shown in Figure 2. The backbone curve defines the boundaries within which the hysteretic response of the component/structure is confined." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 6)

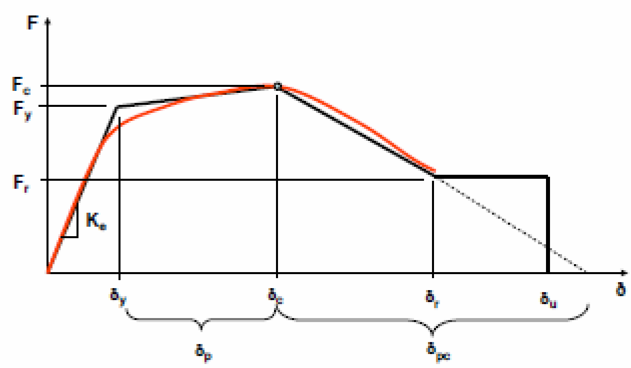


Figure 2. Modified Ibarra-Krawinkler deterioration model. Backbone curve and basic modes of cyclic deterioration (Lignos and Krawinkler, 2009)

(Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 6)

"The quantities *F* and δ are generic force and deformation quantities. For plastic hinge regions $F = M$ and $\delta = \theta$. For SDOF configurations such as wall structures *F* is the story shear force and the deformation quantity δ is the story drift ratio δ/h, denoted θ from here on. … The ultimate deformation capacity δ_u is usually associated with a sudden failure mode or with behavior that can no longer be relied upon. The parameters needed to define the backbone curve are shown in Figure 2." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 6)

"The Interactive Interface for Incremental Dynamic Analysis procedure has a library of ground motion sets available for seismic performance evaluation of SDOF systems. The two existing ground motion sets are described as follows,

- A set of 40 ground motions noted as LMSR-N with magnitude 6.5 ≤ *Mw* ≤ 7.5 and rupture distance13.0*km* ≤ *R* ≤ 30*km*. Detailed information on this ground motion set is presented in Medina and Krawinkler, (2003).
- A set of 44 ground motions denoted as FEMA P695 set that represents far field ground motions normalized using the FEMA P695 Appendix A methodology. All ground motions have been scaled to represent a scale factor of 1.0. Detailed information on this ground motion record set can be found in FEMA P695 and Haselton and Deierlein, (2007)." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 10)

"Incremental dynamic analysis (IDA) is a parametric analysis method that is utilized to estimate seismic performance of structural systems. The procedure involves subjecting the structural model to a set of ground motions, each scaled to multiple levels of ground motion intensity in order to produce response curves (IDA curves) parameterized versus intensity level (see Vamvatsikos and Cornell, 2002). The IDA curve relates a selected intensity measure (IM) of the selected ground motion set with an engineering demand parameter (EDP) of the structural system such as relative displacement, story drift ratio or absolute acceleration. The IDA also known as "dynamic pushover" involves a series of dynamic non-linear time history analysis performed under scaled acceleration histories whose IMs are ideally selected to cover the whole range from elastic to nonlinear and finally to collapse of the structure." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 11)

"Figure 6 illustrates a set of 40 IDA curves (i.e. 40 ground motion records) for an SDOF wall structure with a period of 0.30sec. When the curve becomes flat the structural system loses its lateral resistance, i.e. collapse occurs." (Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 11)

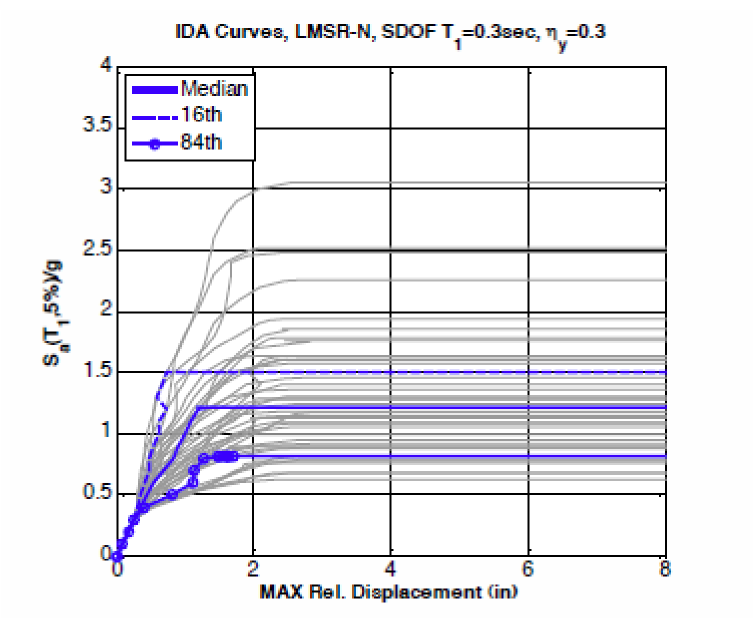


Figure 6. Incremental dynamic analysis curves for an SDOF wall structure

(Interactive Interface for Incremental Dynamic Analysis Procedure, IIIDAP, pg. 12)

QUOTATIONS – HYSTERETIC MODELS THAT INCORPORATE STRENGTH AND STIFFNESS DETERIORATION (2005 IBARRA, MEDINA, KRAWINKLER)

EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng Struct. Dyn. 2005; 34:1489–1511

Hysteretic models that incorporate strength and stiffness deterioration

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"The cyclic deterioration in excursion *i* is defined by the parameter β_i , which is given by the following expression:

$$\beta_i = \left(\frac{E_i}{E_t - \sum_j^i E_j} \right)^c \tag{1}$$

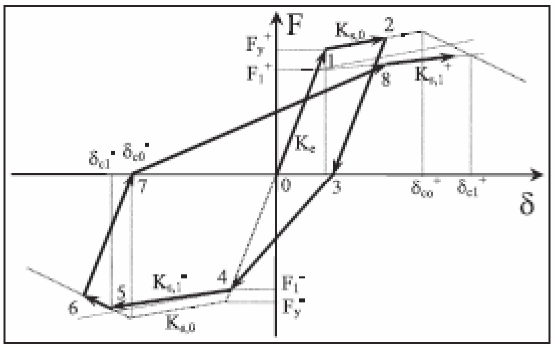
E_i is the hysteretic energy dissipated in excursion *i*, $\sum E_j$ the hysteretic energy dissipated in all previous excursions through loading in both positive and negative directions, E_t the reference hysteretic energy dissipation capacity, $E_t = \gamma F_y \delta_y$. The parameter γ expresses the hysteretic energy dissipation capacity as a function of twice the elastic strain energy at yielding $\left(F_y \delta_y\right)$, it is calibrated from experimental results and can be different for each deterioration mode. Finally, c is the exponent defining the rate of deterioration. Rahnama and Krawinkler [7] suggest that a reasonable range for *c* is between 1.0 and 2.0. If the displacement history consists of constant amplitude cycles, a unit value for *c* implies an almost constant rate of deterioration. For the same displacement history, a value *c = 2* slows down the rate of deterioration in early cycles and accelerates the rate of deterioration in later cycles [7]." (pgs. 1494-1495)

2.3.1. Basic strength deterioration. "<u>It is defined by translating the strain hardening branch toward the origin by an amount equivalent to reducing the yield strength to</u>

$$F_i^+ = \left(1-\beta_{s,i}\right) F_{i-1}^+ \text{ and } F_i^- = \left(1-\beta_{s,i}\right) F_{i-1}^- \tag{3}$$

$F_i^{+/-}$ and $F_{i-1}^{+/-}$ are the deteriorated yield strength after and before excursion *i* respectively.” (pg. 1495) [Underline by Hart]

“A peak-oriented model is used in Figure 6(a) to illustrate the basic strength deterioration mode. At point 3, β_s is calculated for first time and the yield strength on the negative side is reduced from F_y^- to F_1^-.” (pg. 1495) [Underline by Hart]



(a)

Figure 6. Individual deterioration modes, illustrated on a peak-oriented model: (a) basic strength deterioration… (pg. 1496)

7. Rahnama M, Krawinkler H. Effects of soft soil and hysteresis model on seismic demands. John A. Blume Earthquake Engineering Center Report No. 108. Department of CEE, Stanford University, 1993.

APPENDIX 5 STRUCTURAL RELIABILITY CONSIDERATION

The Reliability (Safety) Index used for the shear limit state in the Long Wall is 4.0 compared to 3.5. The 4.0 value is at the upper end of the 3.5 to 4.0 range of values for the Reliability Index used is building code procedure development. The following quotation is from the concrete design book co-authored by Wight and MacGregor:

> "Based on current design practice, β is taken between 3 and 3.5 for ductile failures with average consequences of failure and between 3.5 and 4 for sudden failures or failures having serious consequences [2-7], [2-8]."
>
> [2-7] James G. MacGregor, "Safety and Limit States Design for Reinforced Concrete," *Canadian Journal of Civil Engineering*, Vol. 3, No. 4, December 1976, pp. 484-513.
>
> [2-8] Bruce Ellingwood, Theodore Galambos, James MacGregor, and C. Allan Cornell, *Development of a Probability Based Load Criterion for American National Standard A58*, NBC Special Publication 577, National Bureau of Standards, US Department of Commerce, Washington, DC, June 1980, 222 pp.

From LATB-1, define

Design Demand = $\gamma \bar{D}$ (10.2-28)

Design Capacity = $\phi \bar{C}$ (10.2-29)

Defining the Capacity Reduction Factor as

$$\phi = \exp\left[-0.75\beta\rho_C\right] \quad (10.2\text{-}25)$$

and the Load Amplification Factor as

$$\gamma = \exp\left[0.75\beta\rho_D\right] \quad (10.2\text{-}26)$$

it follows that

$$\gamma\bar{D} = \phi\bar{C} \quad (10.2\text{-}27)$$

It is worth going back now that the math is done and looking first at Equations (10.2-26) and (10.2-28) and then at Equations (10.2-25) and (10.2-29). We see that β is present in both γ and ϕ, but γ is only a function of ρ_D, i.e. not ρ_C, and ϕ is only a function of ρ_C and not ρ_D.

Figure 10.2-1 shows a plot of ϕ versus β and ρ_C. Figure 10.2-2 shows a plot of γ versus β and ρ_D.

When looking at these two figures, recall that in classical code design format, the Capacity is calculated using less than expected values in many cases; thus a direct comparison of the results in these figures would require an in-depth study of the basis for the code equations and also the parameter values in the equations. A similar comparison would be required for Demand.

Performance Based Design for Frequent Earthquakes and Wind Loads typically considers β values of one or less. Consider the curves for $\beta = 0.25$ in Figure 10.2-1 and 10.2-2. The ϕ values are always greater than 0.9 and the γ values are always less than 1.1. But if $\beta = 1.0$, then these limits for ϕ and γ change to ≈ 0.7 and 1.5, respectively. This shows for Frequent natural hazard exposure that the direct benefit to the client in reducing uncertainty in testing, analysis and construction quality control. This observation and conclusion is the same for $\beta = 3$ or $\beta = 4$.

Now let's look at design using the Central Safety Factor pair of glasses. Equation (10.2-27) can be rearranged and expressed in terms of the Central Safety Factor and becomes

$$\beta_C = \frac{\bar{C}}{\bar{D}} = \frac{\gamma}{\phi} \qquad (10.2\text{-}30)$$

This equation provides the structural engineer with, for a target value of β, the minimum value of the Central Safety Factor if failure of the limit state is to be avoided. Figures 10.2-3 to 10.2-6 show a plot of the Central Safety Factor for different values of Reliability Index, Coefficient of Variation of Demand, and Coefficient of Variation of Capacity. Tables 10.2-1 to 10.2-6 provide values for the Central Safety Factor using Equations (10.2-25), (10.2-26), and (10.2-30). I have selected the ρ_D values of 10%, 20%, 35% and 50% to correspond to the classification levels of Superior, Good, Fair and Poor from Table 1.4-1. If we look at Figure 10.2-4 on Table 10.2-4 and select a value of ρ_C = 20%, i.e. Good, then for β = 3 the value of the Central Safety Factor is approximately 2.5.

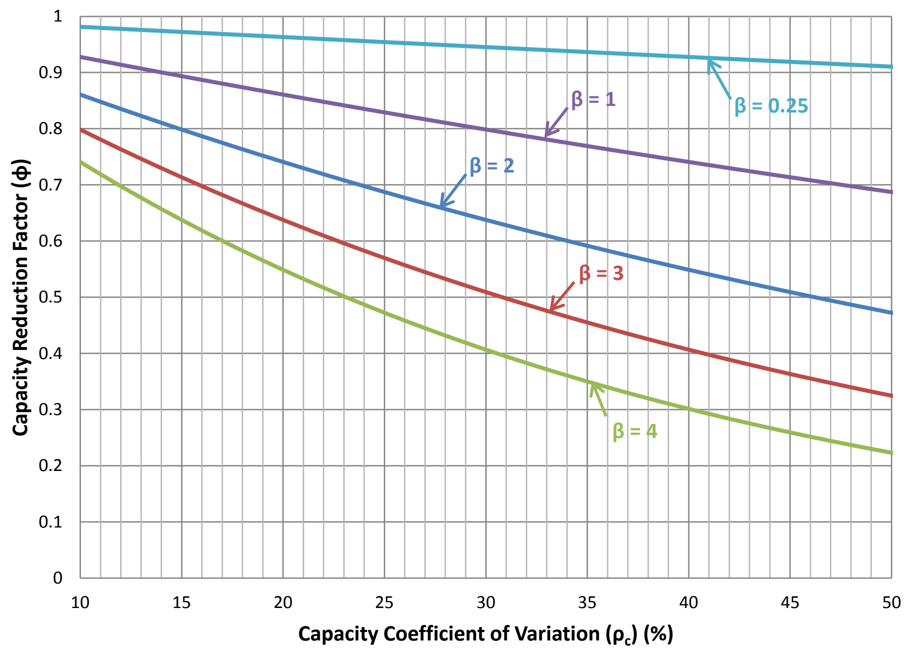


Figure 10.2-1 Capacity Reduction Factor as a Function of Reliability Index and Capacity Coefficient of Variation (Log-Normal)

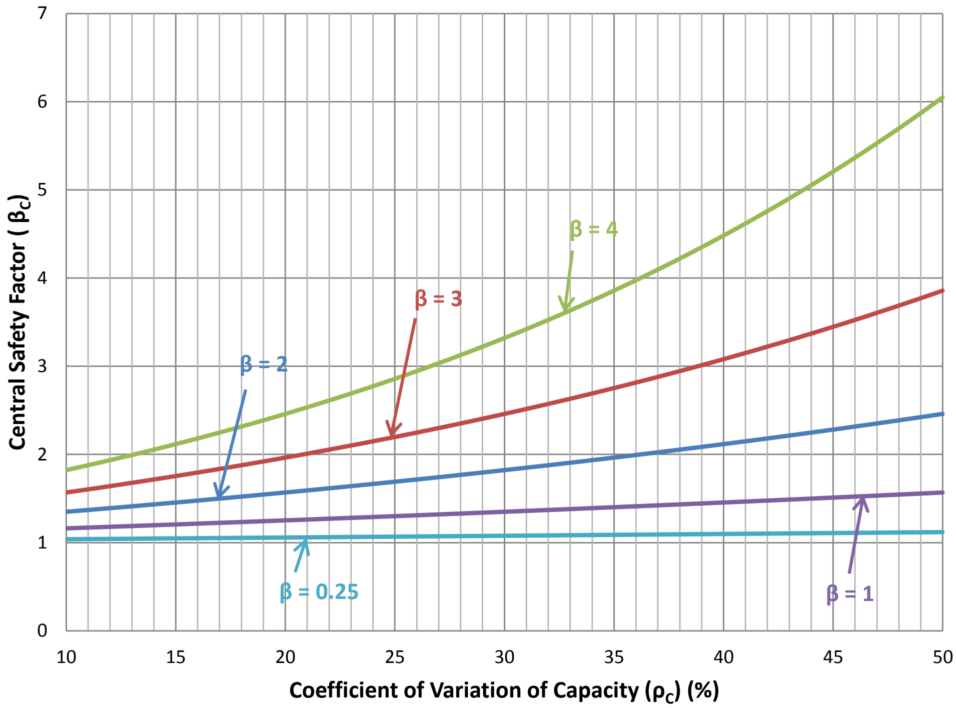


Figure 10.2-3 Central Safety Factor for $\rho_D = 10\%$ (Log-Normal)

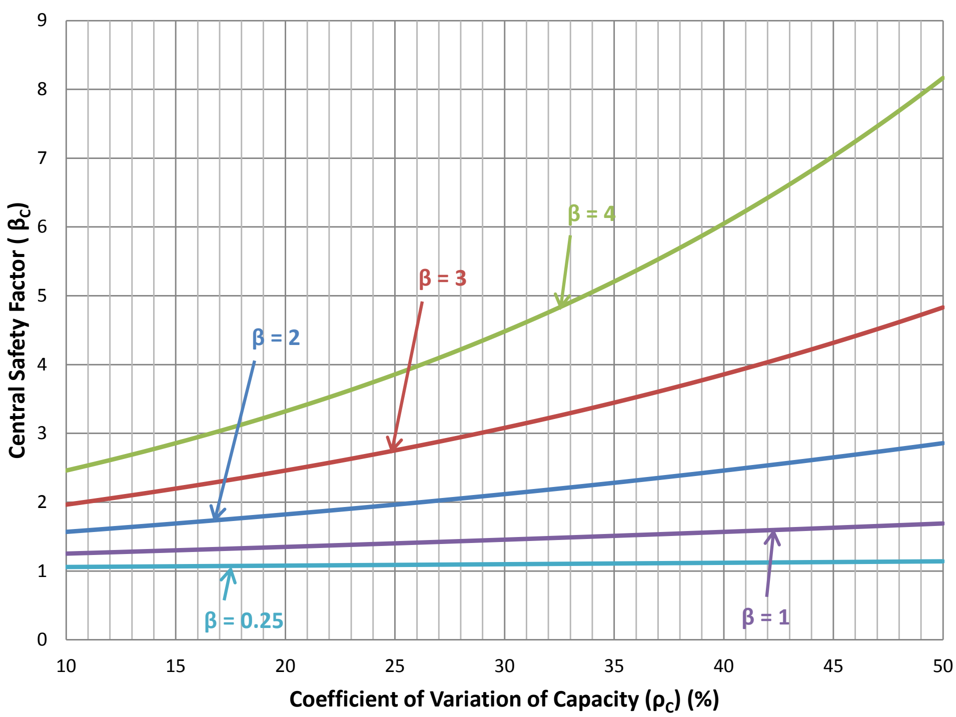


Figure 10.2-4 Central Safety Factor for $\rho_D = 20\%$ (Log-Normal)

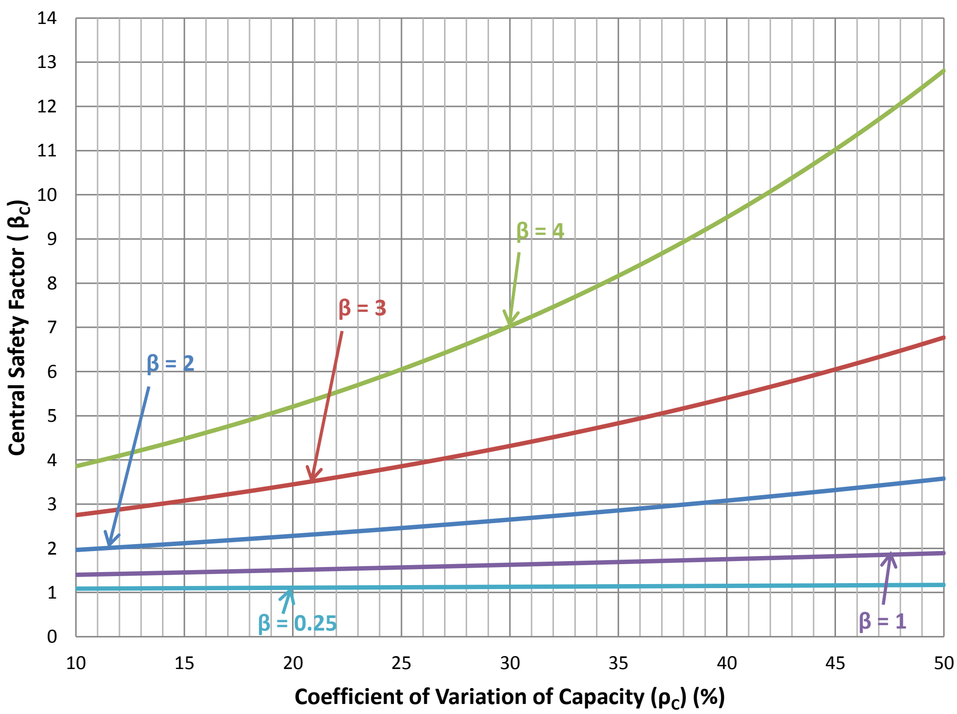


Figure 10.2-5 Central Safety Factor for $\rho_D = 35\%$ (Log-Normal)

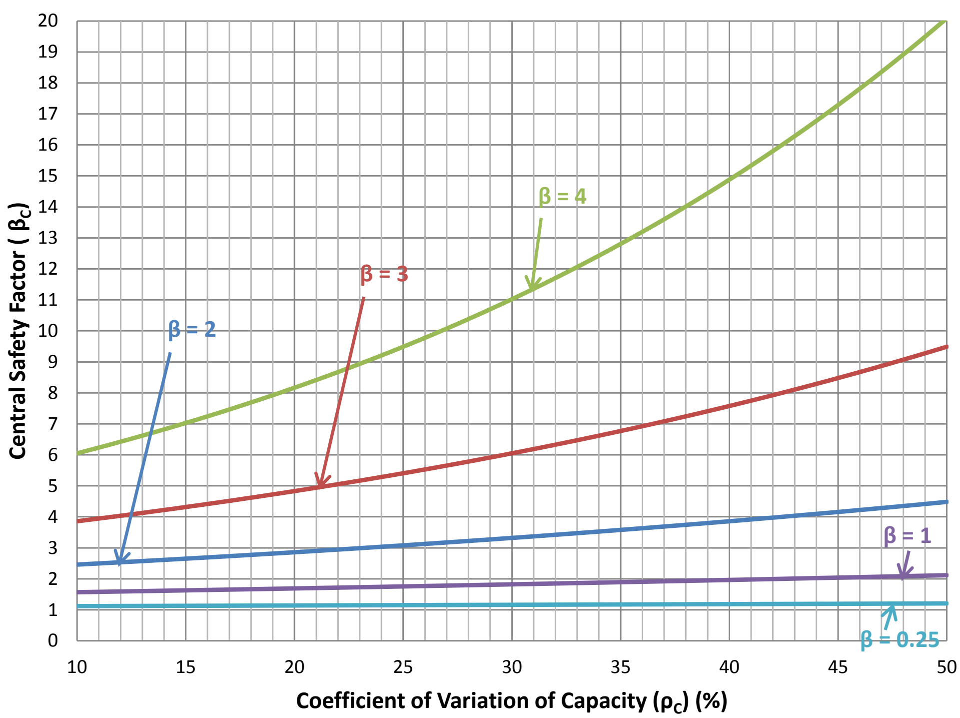


Figure 10.2-6 Central Safety Factor for $\rho_D = 50\%$ (Log-Normal)

Table 10.2-5 Central Safety Factor for Reliability Index of 3.5 (Log-Normal)

Coefficient of Variation of Demand (%)	Coefficient of Variation of Capacity (%)					
	10	15	20	25	30	35
10	1.69	1.93	2.20	2.51	2.86	3.26
15	1.93	2.20	2.51	2.86	3.26	3.72
20	2.20	2.51	2.86	3.26	3.72	4.24
25	2.51	2.86	3.26	3.72	4.24	4.83
30	2.86	3.26	3.72	4.24	4.83	5.51
35	3.26	3.72	4.24	4.83	5.51	6.28
40	3.72	4.24	4.83	5.51	6.28	7.16

Table 10.2-6 Central Safety Factor for Reliability Index of 4 (Log-Normal)

Coefficient of Variation of Demand (%)	Coefficient of Variation of Capacity (%)					
	10	15	20	25	30	35
10	1.82	2.12	2.46	2.86	3.32	3.86
15	2.12	2.46	2.86	3.32	3.86	4.48
20	2.46	2.86	3.32	3.86	4.48	5.21
25	2.86	3.32	3.86	4.48	5.21	6.05
30	3.32	3.86	4.48	5.21	6.05	7.03
35	3.86	4.48	5.21	6.05	7.03	8.17
40	4.48	5.21	6.05	7.03	8.17	9.49