FEMA 307

EVALUATION OF EARTHQUAKE DAMAGED CONCRETE AND MASONRY WALL BUILDINGS

Technical Resources

1998

Applied Technology Council

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Preface

Following the two damaging California earthquakes in 1989 (Loma Prieta) and 1994 (Northridge), many concrete wall and masonry wall buildings were repaired using federal disaster assistance funding. The repairs were based on inconsistent criteria, giving rise to controversy regarding criteria for the repair of cracked concrete and masonry wall buildings. To help resolve this controversy, the Federal Emergency Management Agency (FEMA) initiated a project on evaluation and repair of earthquake damaged concrete and masonry wall buildings in 1996. The project was conducted through the Partnership for Response and Recovery (PaRR), a joint venture of Dewberry & Davis of Fairfax, Virginia, and Woodward-Clyde Federal Services of Gaithersburg, Maryland. The Applied Technology Council (ATC), under subcontract to PaRR, was responsible for developing technical criteria and procedures (the ATC-43 project).

The ATC-43 project addresses the investigation and evaluation of earthquake damage and discusses policy issues related to the repair and upgrade of earthquake-damaged buildings. The project deals with buildings whose primary lateral-force-resisting systems consist of concrete or masonry bearing walls with flexible or rigid diaphragms, or whose vertical-load-bearing systems consist of concrete or steel frames with concrete or masonry infill panels. The intended audience is design engineers, building owners, building regulatory officials, and government agencies.

The project results are reported in three documents. The FEMA 306 report, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual,* provides guidance on evaluating damage and analyzing future performance. Included in the document are component damage classification guides, and test and inspection guides. FEMA 307, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Technical Resources*, contains supplemental information including results from a theoretical analysis of the effects of prior damage on single-degree-of-freedom mathematical models, additional background information on the component guides, and an example of the application of the basic procedures. FEMA 308, *The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, discusses the policy issues pertaining to the repair of earthquake damaged buildings and illustrates how the procedures developed for the project can be used to provide a technically sound basis for policy decisions. It also provides guidance for the repair of damaged components.

The project also involved a workshop to provide an opportunity for the user community to review and comment on the proposed evaluation and repair criteria. The workshop, open to the profession at large, was held in Los Angeles on June 13, 1997 and was attended by 75 participants.

The project was conducted under the direction of ATC Senior Consultant Craig Comartin, who served as Co-Principal Investigator and Project Director. Technical and management direction were provided by a Technical Management Committee consisting of Christopher Rojahn (Chair), Craig Comartin (Co-Chair), Daniel Abrams, Mark Doroudian, James Hill, Jack Moehle, Andrew Merovich (ATC Board Representative), and Tim McCormick. The Technical Management Committee created two Issue Working Groups to pursue directed research to document the state of the knowledge in selected key areas: (1) an Analysis Working Group, consisting of Mark Aschheim (Group Leader) and Mete Sozen (Senior Consultant) and (2) a Materials Working Group, consisting of Joe Maffei (Group Leader and Reinforced Concrete Consultant), Greg Kingsley (Reinforced Masonry Consultant), Bret Lizundia (Unreinforced Masonry Consultant), John Mander (Infilled Frame Consultant), Brian Kehoe and other consultants from Wiss, Janney, Elstner and Associates (Tests, Investigations, and Repairs Consultant). A Project Review Panel provided technical overview and guidance. The Panel members were Gregg Borchelt, Gene Corley, Edwin Huston, Richard Klingner, Vilas Mujumdar, Hassan Sassi, Carl Schulze, Daniel Shapiro, James Wight, and Eugene Zeller. Nancy Sauer and Peter Mork provided technical editing and report production services, respectively. Affiliations are provided in the list of project participants.

The Applied Technology Council and the Partnership for Response and Recovery gratefully acknowledge the cooperation and insight provided by the FEMA Technical Monitor, Robert D. Hanson.

Tim McCormick
PaRR Task Manager

Christopher Rojahn
ATC-43 Principal Investigator
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Prologue

This document is one of three to result from the ATC-43 project funded by the Federal Emergency Management Agency (FEMA). The goal of the project is to develop technically sound procedures to evaluate the effects of earthquake damage on buildings with primary lateral-force-resisting systems consisting of concrete or masonry bearing walls or infilled frames. The procedures are based on the knowledge derived from research and experience in engineering practice regarding the performance of these types of buildings and their components. The procedures require thoughtful examination and review prior to implementation. The ATC-43 project team strongly urges individual users to read all of the documents carefully to form an overall understanding of the damage evaluation procedures and repair techniques.

Before this project, formalized procedures for the investigation and evaluation of earthquake-damaged buildings were limited to those intended for immediate use in the field to identify potentially hazardous conditions. ATC-20, *Procedures for Postearthquake Safety Evaluation of Buildings*, and its addendum, ATC-20-2 (ATC, 1989 and 1995) are the definitive documents for this purpose. Both have proven to be extremely useful in practical applications. ATC-20 recognizes and states that in many cases, detailed structural engineering evaluations are required to investigate the implications of earthquake damage and the need for repairs. This project provides a framework and guidance for those engineering evaluations.

What have we learned?

The project team for ATC-43 began its work with a thorough review of available analysis techniques, field observations, test data, and emerging evaluation and design methodologies. The first objective was to understand the effects of damage on future building performance. The main points are summarized below.

- **Component behavior controls global performance.**

  Recently developed guidelines for structural engineering seismic analysis and design techniques focus on building displacement, rather than forces as the primary parameter for the characterization of seismic performance. This approach models the building as an assembly of its individual components. Force-deformation properties (e.g., elastic stiffness, yield point, ductility) control the behavior of wall panels, beams, columns, and other components. The component behavior, in turn, governs the overall displacement of the building and its seismic performance. Thus, the evaluation of the effects of damage on building performance must concentrate on how component properties change as a result of damage.

- **Indicators of damage (e.g., cracking, spalling) are meaningful only in light of the mode of component behavior.**

  Damage affects the behavior of individual components differently. Some exhibit ductile modes of post-elastic behavior, maintaining strength even with large displacements. Others are brittle and lose strength abruptly after small inelastic displacements. The post-elastic behavior of a structural component is a function of material properties, geometric proportions, details of construction, and the combination of demand actions (axial, flexural, shearing, torsional) imposed upon it. As earthquake shaking imposes these actions on components, the components tend to exhibit predominant modes of behavior as damage occurs. For example, if earthquake shaking and its associated inertial forces and frame distortions cause a reinforced concrete wall panel to rotate at each end, statics defines the relationship between the associated bending moments and shear force. The behavior of the panel depends on its strength in flexure relative to that in shear. Cracks and other signs of damage must be interpreted in the context of the mode of component behavior. A one-eighth-inch crack in a wall panel on the verge of brittle shear failure is a very serious condition. The same size crack in a flexurally-controlled panel may be insignificant with regard to future seismic performance. This is, perhaps, the most important finding of the ATC-43 project: the significance of cracks and other signs of damage, with respect to the future performance of a building, depends on the mode of behavior of the components in which the damage is observed.

• **Damage may reveal component behavior that differs from that predicted by evaluation and design methodologies.**

When designing a building or evaluating an undamaged building, engineers rely on theory and their own experience to visualize how earthquakes will affect the structure. The same is true when they evaluate the effects of actual damage after an earthquake, with one important difference. If engineers carefully observe the nature and extent of the signs of the damage, they can greatly enhance their insight into the way the building actually responded to earthquake shaking. Sometimes the actual behavior differs from that predicted using design equations or procedures. This is not really surprising, since design procedures must account conservatively for a wide range of uncertainty in material properties, behavior parameters, and ground shaking characteristics. Ironically, actual damage during an earthquake has the potential for improving the engineer's knowledge of the behavior of the building. When considering the effects of damage on future performance, this knowledge is important.

• **Damage may not significantly affect displacement demand in future larger earthquakes.**

One of the findings of the ATC-43 project is that prior earthquake damage does not affect maximum displacement response in future, larger earthquakes in many instances. At first, this may seem illogical. Observing a building with cracks in its walls after an earthquake and visualizing its future performance in an even larger event, it is natural to assume that it is worse off than if the damage had not occurred. It seems likely that the maximum displacement in the future, larger earthquake would be greater than if it had not been damaged. Extensive nonlinear time-history analyses performed for the project indicated otherwise for many structures. This was particularly true in cases in which significant strength degradation did not occur during the prior, smaller earthquake. Careful examination of the results revealed that maximum displacements in time histories of relatively large earthquakes tended to occur after the loss of stiffness and strength would have taken place even in an undamaged structure. In other words, the damage that occurs in a prior,

smaller event would have occurred early in the subsequent, larger event anyway.

What does it mean?

The ATC-43 project team has formulated performance-based procedures for evaluating the effects of damage. These can be used to quantify losses and to develop repair strategies. The application of these procedures has broad implications.

• **Performance-based damage evaluation uses the actual behavior of a building, as evidenced by the observed damage, to identify specific deficiencies.**

The procedures focus on the connection between damage and component behavior and the implications for estimating actual behavior in future earthquakes. This approach has several important benefits. First, it provides a meaningful engineering basis for measuring the effects of damage. It also identifies performance characteristics of the building in its pre-event and damaged states. The observed damage itself is used to calibrate the analysis and to improve the building model. For buildings found to have unacceptable damage, the procedures identify specific deficiencies at a component level, thereby facilitating the development of restoration or upgrade repairs.

• **Performance-based damage evaluation provides an opportunity for better allocation of resources.**

The procedures themselves are technical engineering tools. They do not establish policy or prescribe rules for the investigation and repair of damage. They may enable improvements in both private and public policy, however. In past earthquakes, decisions on what to do about damaged buildings have been hampered by a lack of technical procedures to evaluate the effects of damage and repairs. It has also been difficult to investigate the risks associated with various repair alternatives. The framework provided by performance-based damage evaluation procedures can help to remove some of these roadblocks. In the long run, the procedures may tend to reduce the prevailing focus on the loss caused by damage from its pre-event conditions and to increase the focus on what the damage reveals about future building performance. It makes little

sense to implement unnecessary repairs to buildings that would perform relatively well even in a damaged condition. Nor is it wise to neglect buildings in which the component behavior reveals serious hazards regardless of the extent of damage.

- **Engineering judgment and experience are essential to the successful application of the procedures.**

  ATC-20 and its addendum, ATC-20-2, were developed to be used by individuals who might be somewhat less knowledgeable about earthquake building performance than practicing structural engineers. In contrast, the detailed investigation of damage using the performance-based procedures of this document and the companion FEMA 306 report (ATC, 1998a) and FEMA 308 report (ATC, 1998b) must be implemented by an experienced engineer. Although the documents include information in concise formats to facilitate field operations, they must not be interpreted as a "match the pictures" exercise for unqualified observers. Use of these guideline materials requires a thorough understanding of the underlying theory and empirical justifications contained in the documents. Similarly, the use of the simplified direct method to estimate losses has limitations. The decision to use this method and the interpretation of the results must be made by an experienced engineer.

- **The new procedures are different from past damage evaluation techniques and will continue to evolve in the future.**

  The technical basis of the evaluation procedures is essentially that of the emerging performance-based

seismic and structural design procedures. These will take some time to be assimilated in the engineering community. The same is true for building officials. Seminars, workshops, and training sessions are required not only to introduce and explain the procedures but also to gather feedback and to improve the overall process. Additionally, future materials-testing and analytical research will enhance the basic framework developed for this project. Current project documents are initial editions to be revised and improved over the years.

In addition to the project team, a Project Review Panel has reviewed the damage evaluation and repair procedures and each of the three project documents. This group of experienced practitioners, researchers, regulators, and materials industry representatives reached a unanimous consensus that the products are technically sound and that they represent the state of knowledge on the evaluation and repair of earthquake-damaged concrete and masonry wall buildings. At the same time, all who contributed to this project acknowledge that the recommendations depart from traditional practices. Owners, design professionals, building officials, researchers, and all others with an interest in the performance of buildings during earthquakes are encouraged to review these documents and to contribute to their continued improvement and enhancement. Use of the documents should provide realistic assessments of the effects of damage and valuable insight into the behavior of structures during earthquakes. In the long run, they hopefully will contribute to sensible private and public policy regarding earthquake-damaged buildings.

1. Introduction

1.1 Purpose And Scope

The purpose of this document is to provide supplemental information for evaluating earthquake damage to buildings with primary lateral-force-resisting systems consisting of concrete and masonry bearing walls and infilled frames. This document includes background and theoretical information to be used in conjunction with the practical evaluation guidelines and criteria given in *FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Basics Procedures Manual* (ATC, 1998a). In both documents, concrete and masonry wall buildings include those with vertical-load-bearing wall panels, with and without intermediate openings. In these documents, shear wall buildings also include those with vertical-load-bearing frames of concrete or steel that incorporate masonry or concrete infill panels to resist horizontal forces. The FEMA 306 procedures for these building types address:

a. The investigation and documentation of damage caused by earthquakes.

b. The classification of the damage to building components, according to mode of structural behavior and severity.

c. The evaluation of the effects of the damage on the performance of the building during future earthquakes.

d. The development of hypothetical measures that would restore the performance to that of the undamaged building.

Supplemental data in this document, FEMA 307, includes the results of the efforts of two issues working groups that focused on the key aspects of adapting and enhancing existing technology for the purposes of the evaluation and repair of earthquake-damaged buildings. The general scope of work for each group is briefly outlined in the following two sections.

1.2 Materials Working Group

The Materials Working Group effort was a part of the overall ATC-43 project. The primary objectives of the Materials Working Group were:

a. To summarize tests and investigative techniques that can be used to document and evaluate existing structural conditions, particularly the effects of earthquake damage, in concrete and masonry wall buildings.

b. To recommend modifications to component force-deformation relationships currently used in nonlinear structural analysis, based on the documented effects of damage similar to that caused by earthquakes.

c. To describe the specification and efficacy of methods for repair of component damage in a coordinated format suitable for inclusion in the final document.

Figure 1-1 illustrates the idealization of the force-deformation relationships from actual structural component hysteretic data for use in nonlinear analysis. The focus of the Materials Working Group was the generalized force-deformation relationship for structural components of concrete and masonry wall buildings, shown in Figure 1-2.

1.2.1 Tests and Investigations

The scope included review of experimental and analytical research reports, technical papers, standards, and manufacturers' specifications. Practical example applications relating to the documentation, measurement, and quantification of the structural condition of concrete and masonry walls and in-fill frame walls were also reviewed. The reviews focused on tests and investigative techniques for identifying and evaluating cracking, crushing, deterioration, strength, and general quality of concrete or masonry and yielding, fracture, deterioration, strength, and location of reinforcing steel. Based on this review of existing information, practical guidelines for appropriate tests and investigative techniques were developed and are included in FEMA 306. These guidelines consist of outline specifications for equipment, materials, and procedures required to execute the tests, as well as criteria for documenting and interpreting the results.

1.2.2 Component Behavior and Modeling

The members of the group reviewed experimental and analytical research reports, technical papers, and practical example applications relating to the force-deformation behavior of concrete and masonry walls and in-fill frame walls. Of particular interest were the effects of damage of varying nature and extent on the hysteretic characteristics of elements and components
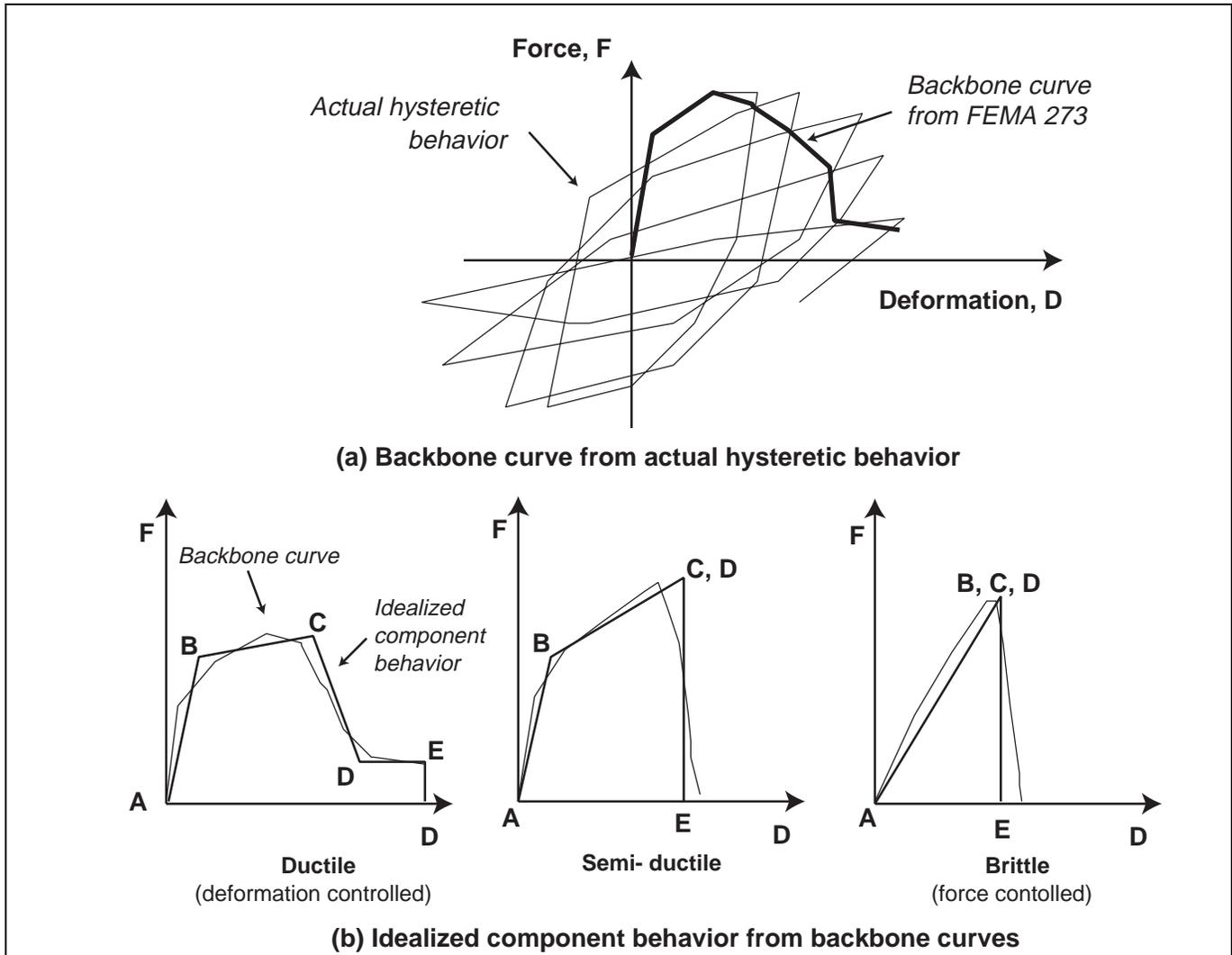
Figure 1-1   *Component Force-Deformation Relationships*

subject to cyclic lateral loads. The types of damage investigated included cracking and crushing of concrete or masonry and yielding and fracture of reinforcing steel. Components included a wide variety of configurations for vertical-load-bearing and infilled-frame elements. Materials included reinforced concrete, reinforced masonry, and unreinforced masonry.

Based on the review, practical guidelines for identifying and modeling the force-deformation characteristics of damaged components were developed and included in FEMA 306. These consist of modifications (B', C', D', E') to the generalized force-deformation relationships for undamaged components, as shown in Figure 1-2. Supplemental information on these modifications is included in this volume in Chapters 2 (Concrete), 3

(Reinforced Masonry), 4 (Unreinforced Masonry), and 5 (Infilled Frames).

1.2.3   Repair Techniques

The Materials Group also reviewed experimental and analytical research reports, technical papers, standards, manufacturers' specifications, and practical example applications relating to the repair of damage in concrete and masonry walls and infilled-frame walls. The primary interest was the repair of earthquake damage to structural components. The review focused on materials and methods of installation and tests of the effectiveness of repair techniques for cracking, crushing, and deterioration of concrete or masonry and yielding, fracture, and deterioration of reinforcing steel.
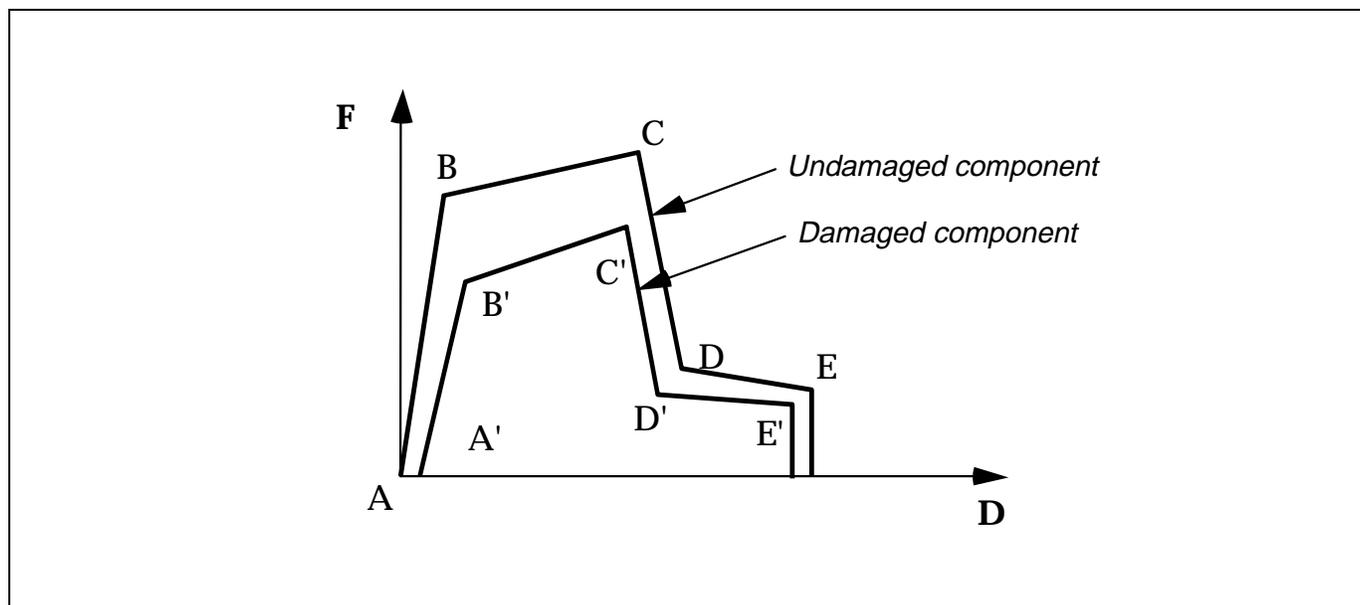
Figure 1-2   ***Generalized Undamaged and Damaged Component Curves***

Based on the review, practical guidelines for damage repair were developed and are contained in *FEMA 308: The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings* (ATC, 1998b). These guidelines consist of outline specifications for equipment, materials, and procedures required to execute the repairs, as well as criteria for quality control and verification of field installations.

1.3   Analysis Working Group

The work of the Analysis Working Group was a sub-project of the overall ATC-43 project. The primary objectives of the group were:

*   To determine whether existing structural analysis techniques are capable of capturing the global effects of previous earthquake damage on future seismic performance

*   To formulate practical guidance for the use of these analysis techniques in design-oriented evaluation and repair of damaged masonry and concrete wall buildings.

Chapter 6 summarizes the results of the Analysis Working Group efforts. Work consisted primarily of analytical studies of representative single-degree-of-freedom (SDOF) oscillators subjected to a range of earthquake ground motions. The study was formulated

so that the following question might be answered (see Figure 1-3): If a building has experienced damage in an earthquake (the *damaging earthquake*), and if that intermediate damage state can be characterized in terms of its effect on the global force-displacement relationship, how will the damage influence global response to a subsequent earthquake (the Performance Earthquake)?

The SDOF oscillators had force-displacement relationships that represent the effects of earthquake damage on the global dynamic response of hypothetical buildings to earthquake ground motions. Types of global force-displacement relationships considered included those shown in Figure 1-4.

The results obtained using existing simplified analyses methods were compared to the time-history results. The group was particularly interested in understanding how nonlinear static analysis methods might be used to represent the findings. Regarding the nonlinear static methods, consideration was given to the applicability of the coefficient method, the capacity-spectrum method, and the secant method of analysis, as summarized in FEMA-273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (ATC, 1997a) and ATC-40 *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996). The work included a study of the accuracy of the various methods in terms of predicting future performance. The study included an assessment of the
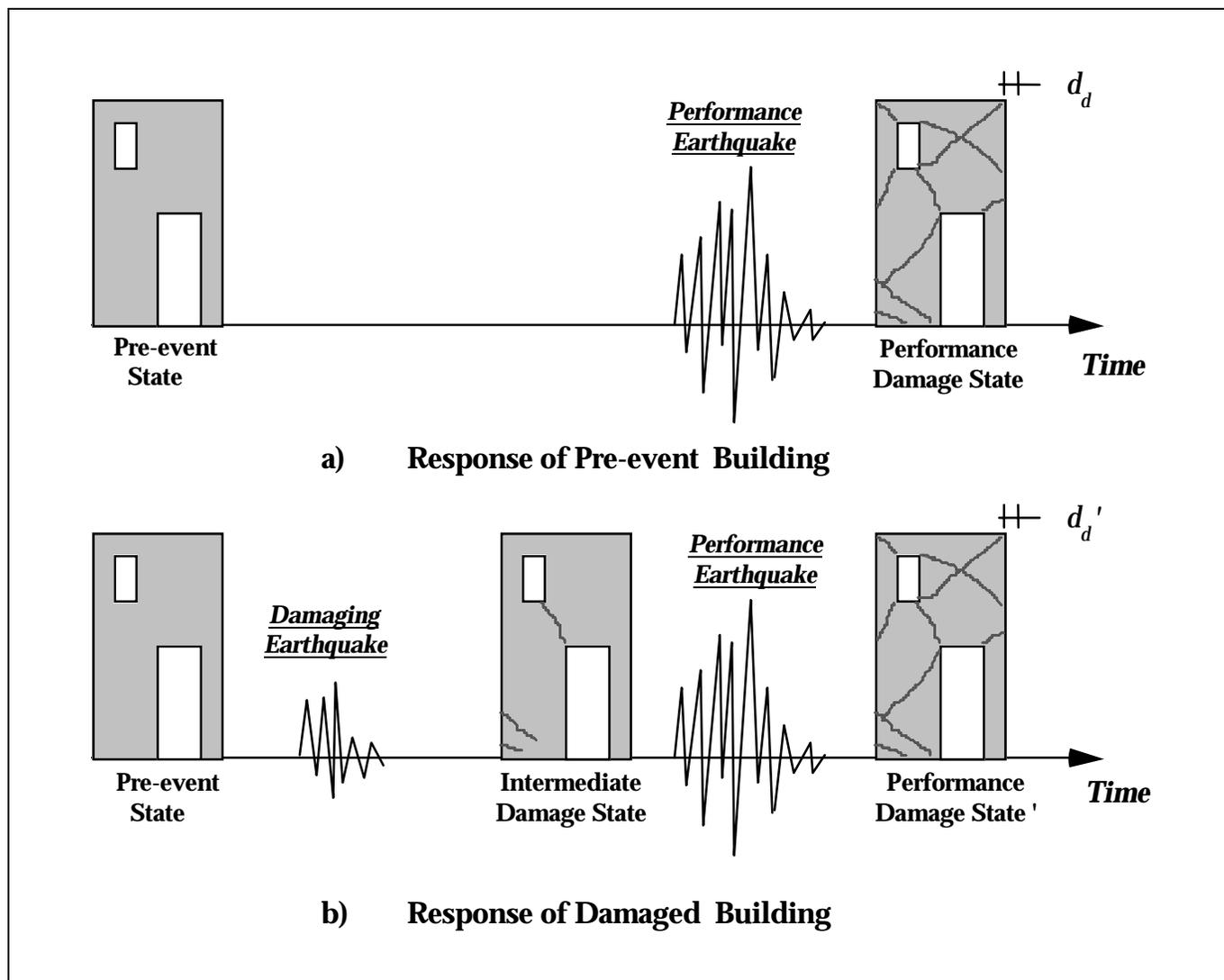
Figure 1-3   *Effect of Damage on Building Response*

sensitivity of the predictions to variations in global load-deformation characteristics and to variations in ground motion characteristics. The results are reflected in the procedures presented in FEMA 306.

1.4 References

ATC, 1996, *The Seismic Evaluation and Retrofit of Concrete Buildings*, Applied Technology Council, ATC-40 Report, Redwood City, California.

ATC, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council (ATC-33 project) for the Building Seismic Safety Council, published by the

Federal Emergency Management Agency, Report No. FEMA 273, Washington, D.C.

ATC, 1997b, NEHRP *Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council (ATC-33 project) for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Report No. FEMA 274, Washington, D.C.

ATC, 1998a, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project) for the Partnership for Response and Recovery, published by the Federal
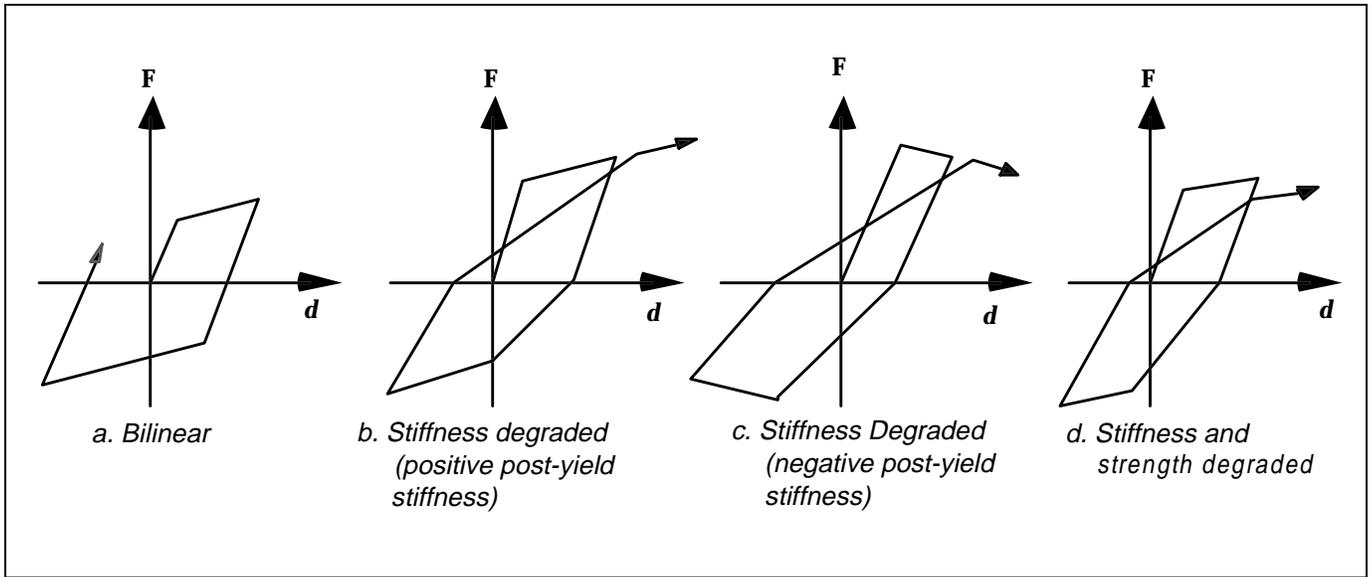
Figure 1-4   ***Global Load-Displacement Relationships***

Emergency Management Agency, Report No. FEMA 306, Washington D.C.

ATC, 1998b, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, prepared by the Applied Technology Council (ATC-43 project) for the Partnership for Response and Recovery, published by the Federal Emergency Management Agency, Report No. FEMA 308, Washington D.C.

2. Reinforced Concrete Components

2.1 Commentary and Discussion

2.1.1 Development of Component Guides and λ Factors

The Component Damage Classification Guides (Component Guides) and component modification factors (λ factors) for reinforced concrete walls were developed based on an extensive review of the research. The main references used are listed in the tabular bibliography of Section 2.3.

2.1.1.1 Identical Test Specimens Subjected to Different Load Histories

As indicated in FEMA 306, the ideal way to establish λ factors would be from structural tests designed specifically for that purpose. Two identical test specimens would be required for each structural component of interest. One specimen would be tested to represent the component in its *post-event* condition subjected to the performance earthquake; the second specimen would be tested to represent the component in its *pre-event* condition subjected to the performance earthquake. The λ values would be derived from the differences in the force-displacement response between the two specimens.

Research to date on reinforced concrete walls does not include test programs as described above. There are only a few tests of identical wall specimens subjected to different loading histories, and typically this is only a comparison of monotonic versus cyclic behavior. For reinforced concrete columns, there are more studies of the effects of load history (El-Bahy et al., 1997; Kawashima and Koyama, 1988) but these studies have not focused on the specific problem of comparing previously damaged components to undamaged components.

2.1.1.2 Interpretation of Individual Tests

In the absence of tests directly designed to develop λ factors, the factors can be inferred from individual cyclic-static tests. This is done by examining the change in force-displacement response from cycle to cycle as displacements are increased. Initial cycles can be considered representative of the damaging earthquake, and subsequent cycles representative of the behavior of an initially damaged component.

The general process of interpreting the test data is outlined in the diagram of Figure 2-1. Each structural test is considered according to the component type and behavior mode represented by the test. At intervals along the load-displacement history of the test the critical damage indicators, such as spalling, cracking, etc., are noted. The damage indicators at each interval are correlated with the displacement ductility reached at that point of the test and with the characteristics of subsequent cycles of the test. From the comparisons of initial and subsequent cycles, λ values are estimated. Critical damage indicators and the associated λ factors are then discretized into different damage severity levels.

The ranges of component displacement ductility, μ_Δ, associated with damage severity levels and λ factors and for each Component Guide are given in Table 2-1. The range of ductility values are the result of the differences in test procedures, specimen details, and relative values of coincident loading (shear, moment, axial load). See the remarks column of Table 2-1 for specific factors affecting individual components. Typical force-displacement hysteresis loops from wall tests are given in Section 2.2. A discussion of the relationship between cracking and damage severity for reinforced masonry is given in Section 3.1.2. This discussion is largely applicable to reinforced concrete as well as reinforced masonry.

In estimating the λ values, it was considered that some stiffness and strength degradation would occur in a structural component in the course of the Performance Earthquake, whether or not it was previously subjected to a damaging earthquake. As discussed in FEMA 306, the λ factors refer to the difference in the stiffness, strength, and displacement capacity of the performance earthquake response, between a pre-event component and a post-event component.

2.1.1.3 Accuracy

The λ factors are considered accurate to one significant digit, as presented in the Component Damage Classification Guides. In the case of component types and behavior modes which are not well covered in the research, engineering judgment and comparisons to similar component types or behavior modes were used
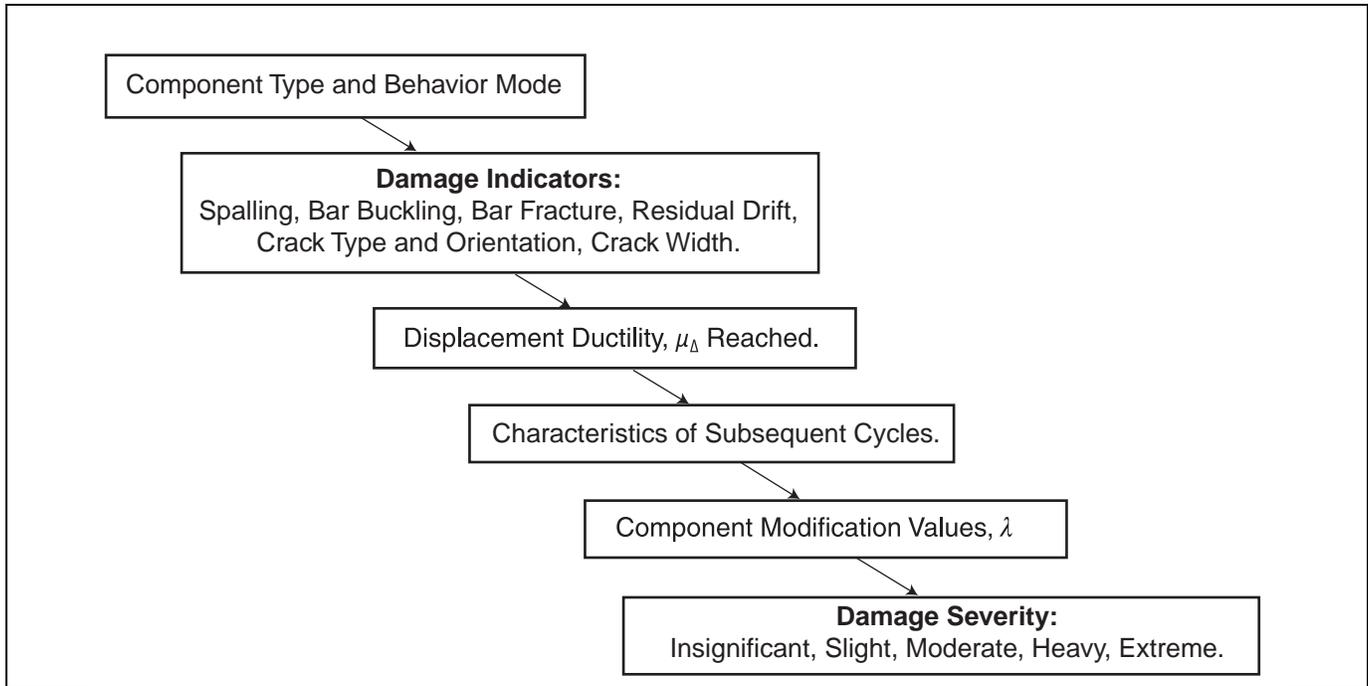
Figure 2-1   *Diagram of process used to develop component guides and component modification factors.*

to establish λ factors. In cases of uncertainty, the recommended λ factors and severity classifications are designed to be conservative — that is, the factors and classifications may overestimate the effect of damage on future performance.

Only limited research is available from which to infer specific λ_D values. However, a number of tests support the general idea that ultimate displacement capacity can be reduced because of previous damaging cycles. Comparisons of monotonic to cyclic-static wall tests show greater displacement capacities for monotonic loading, and Oesterle et al. (1976) conclude, "structural

wall performance under load reversals is a function of load history. The previous level of maximum deformation is critical."

For reinforced concrete columns, Mander et al. (1996) have shown a correlation between strength degradation and cumulative plastic drift. El-Bahy et al, (1997) have shown similar results. This research generally supports the λ_D values recommended for reinforced concrete, which are 0.9 at moderate damage and 0.7 to 0.8 at heavy damage.

Table 2-1   ***Ranges of reinforced concrete component displacement ductility, μ_Δ, associated with damage severity levels and λ factors***

Component Guide	Damage Severity				Remarks on Ductility Ranges
	Insignif.	Slight	Moderate	Heavy	
RC1A Ductile Flexural	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 4-8$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3-10$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Heavy* not used	Slight category will only occur for low axial loads, where concrete does not spall until large ductilities develop
RC1B Flexure/ Diagonal Tension	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Slight* not used	$\mu_\Delta \approx 2-6$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	$\mu_\Delta \approx 2-8$ $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	Ductility depends on ratio of flexural to shear strength. Lower ductility indicates behavior similar to preemptive diagonal tension. Higher ductility indicates behavior similar to ductile flexural.
RC1C Flexure/ Web Crushing	$\mu_\Delta \leq 3$ See RC1B	*Slight* not used	$\mu_\Delta \approx 2-6$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	$\mu_\Delta \approx 3-8$ $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	Ductility depends on ratio of flexural to web crushing strength. Lower ductility indicates behavior similar to preemptive web crushing. Higher ductility indicates behavior similar to ductile flexural.
RC1D Flexure/ Sliding Shear	$\mu_\Delta \leq 3$ See RC1A	$\mu_\Delta \approx 4-6$ See RC1A	*Moderate* not used	$\mu_\Delta \approx 4-8$ $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	Ductility depends on ratio of flexural to sliding shear strength.
RC1E Flexure/ Boundary Compression	$\mu_\Delta \leq 3$ See RC1A	$\mu_\Delta \approx 4-6$ See RC1A	$\mu_\Delta \approx 3-6$ See RC1A	$\mu_\Delta \approx 4-8$ $\lambda_K = 0.4$ $\lambda_Q = 0.6$ $\lambda_D = 0.7$	Slight category will only occur for lower axial loads, where concrete does not spall until large ductilities develop. Lower ductility relates poorer confinement conditions. Higher ductility indicates behavior similar to ductile flexural
RC2A Ductile Flexural	$\mu_\Delta \leq 3$ See RC1A	$\mu_\Delta \approx 4-6$ See RC1A	$\mu_\Delta \approx 3-10$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Heavy* not used	See RC1A
RC2H Preemptive Diagonal Shear	$\mu_\Delta \leq 1$ $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Slight* not used	$\mu_\Delta \leq 1.5$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	$\mu_\Delta \leq 2$ $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	Force controlled behavior associated with low ductility levels.
RC3B Flexure/ Diagonal Tension	$\mu_\Delta \leq 3$ See RC1B	*Slight* not used	$\mu_\Delta \approx 2-6$ See RC1B	$\mu_\Delta \approx 2-8$ $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	See RC1B
RC3D Flexure/ Sliding Shear	$\mu_\Delta \leq 3$ See RC1D	$\mu_\Delta \approx 4-6$ See RC1D	*Moderate* not used	$\mu_\Delta \approx 3-8$ $\lambda_K = 0.2$ $\lambda_Q = 0.3$ $\lambda_D = 0.7$	Sliding shear may occur at lower ductility levels that RC1D because of less axial load.

2.2 Typical Force-Displacement Hysteretic Behavior

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**		
Component Type:	**Isolated Wall or Stronger Wall Pier**	RC1A	**Example 1 of 1**
Predominant Behavior Mode:	**Ductile Flexure**		
Secondary Behavior Mode:	**—**		

Reference: Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)
Specimen: B3

Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$

Damage at +6-in. deflection
$\Delta = 6$ in $\Delta/h_w = 0.033$ $\lambda_Q = 1.0$
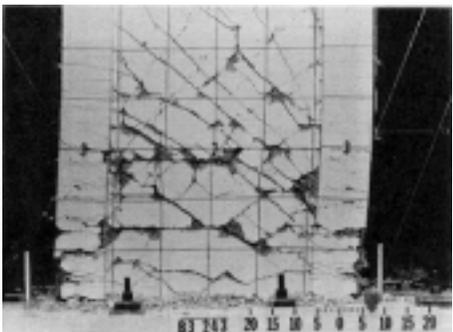
Damage at +8-in. deflection
$\Delta = 8$ in $\Delta/h_w = 0.044$ $\lambda_Q = 0.7$
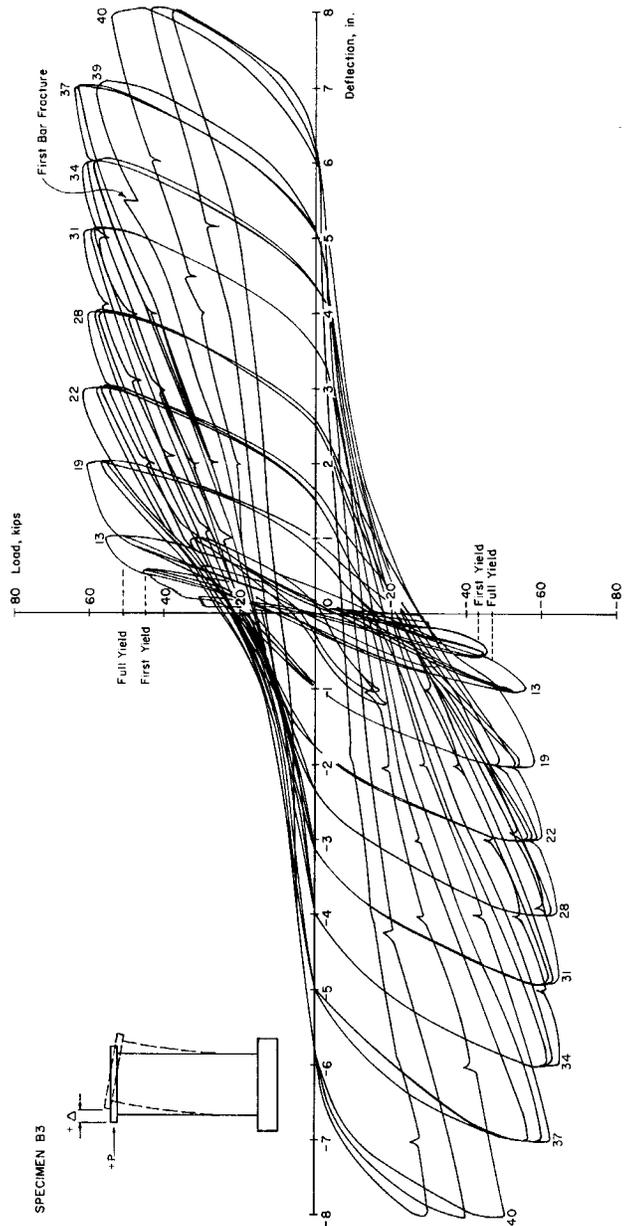
Behavior of Specimen B3
Load versus deflection relationship

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Diagonal Tension**
Secondary Behavior Mode:	**—**

$\boxed{\text{RC1B}}$ **Example 1 of 2**

Reference:	Paulay and Priestley (1992)
Specimen:	Figure 8.3 of reference
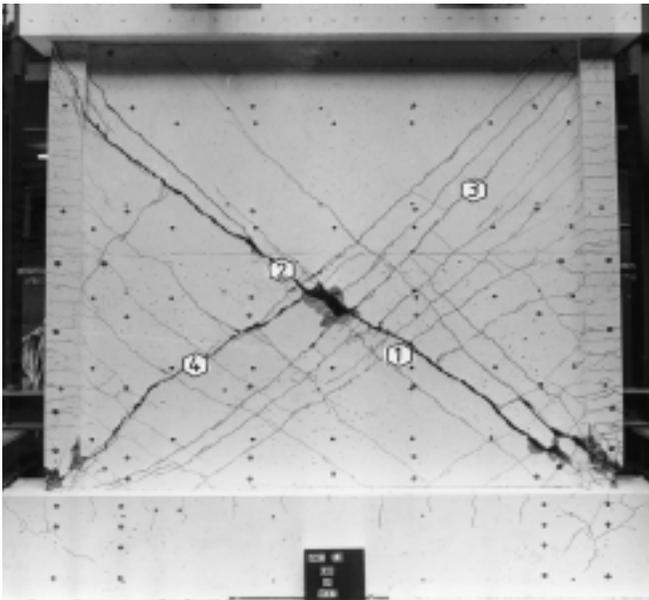
Failure of a squat wall due to diagonal tension after reversed cyclic loading.
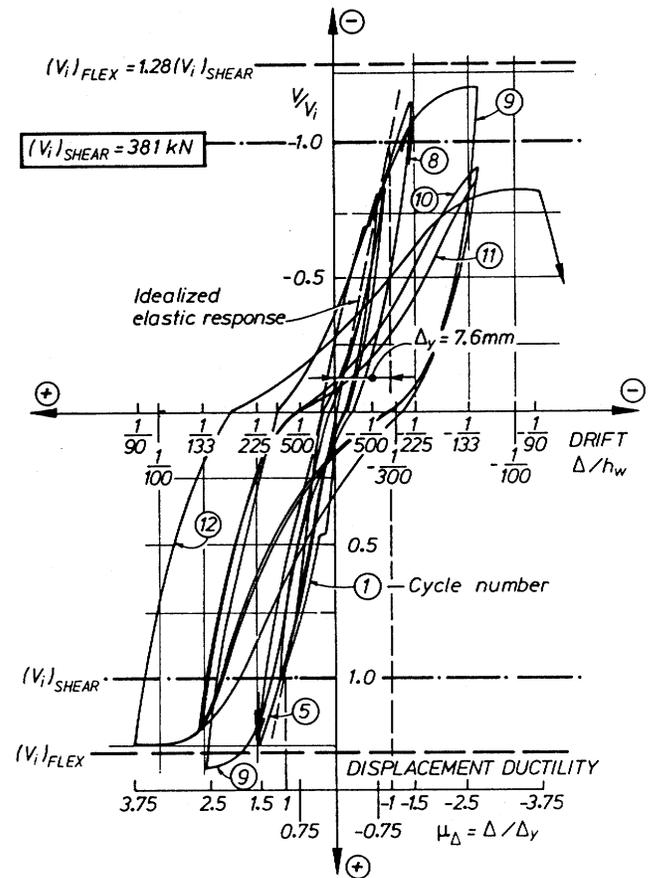
Hysteretic response of a squat wall that eventually failed in shear.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Diagonal Tension**
Secondary Behavior Mode:	**Flexure/Web Crushing**

RC1B   **Example 2 of 2**

Reference:	Shiu et al. (1981)
Specimen:	PW-1
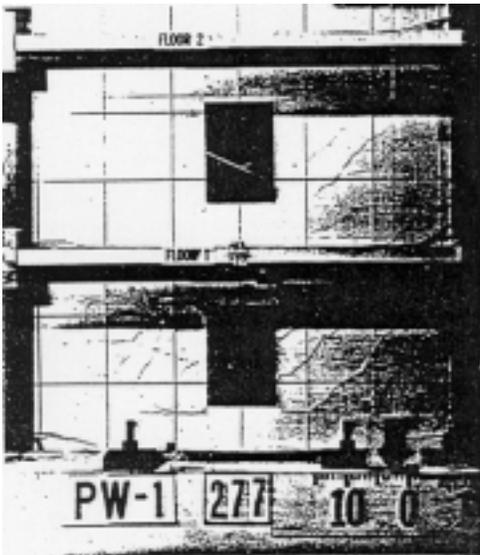
Crack pattern of specimen PW-1 at end of Phase II.
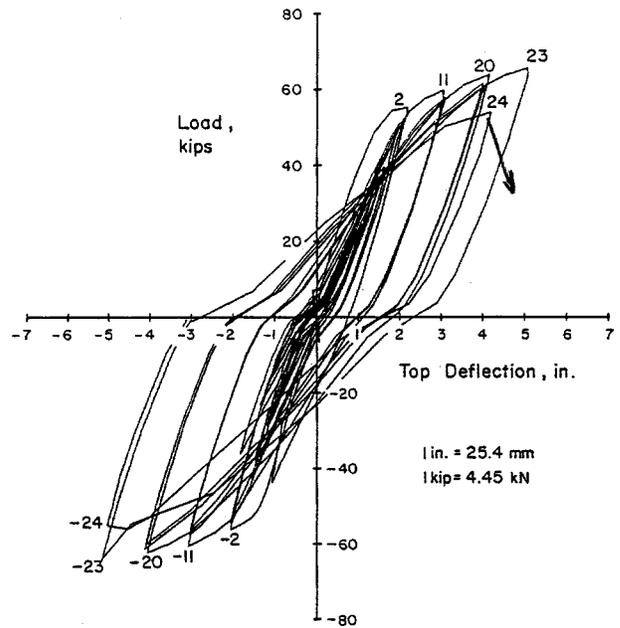
Load versus top deflection relationship for specimen PW-1.
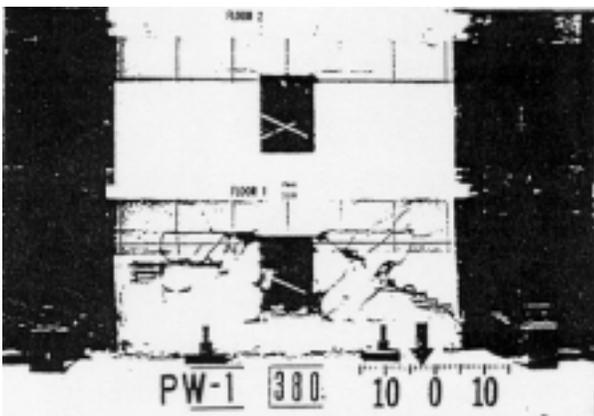
Specimen PW-1 at end of test.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Web Crushing**
Secondary Behavior Mode:	**—**

RC1C   **Example 1 of 3**

Reference: Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)

Specimen: F2
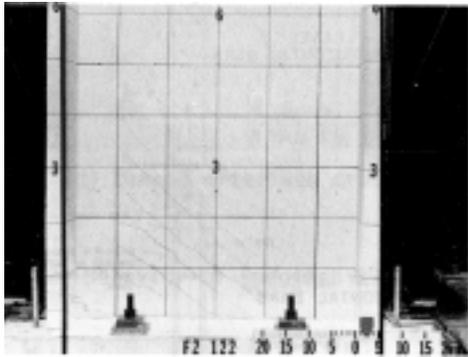
Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
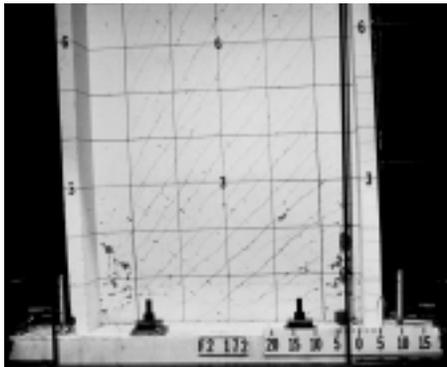
Damage prior to web crushing
$\Delta = 4$ in $\Delta/h_w = 0.022$ $\lambda_Q = 1.0$
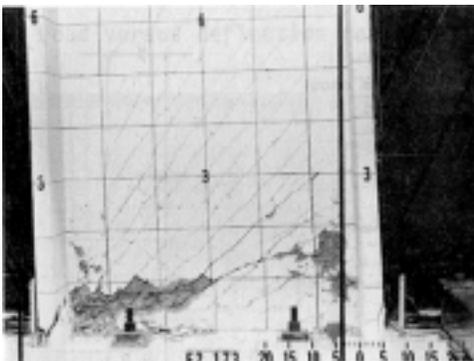
Damage after web crushing
$\Delta = 5$ in $\Delta/h_w = 0.028$ $\lambda_Q = 0.3$
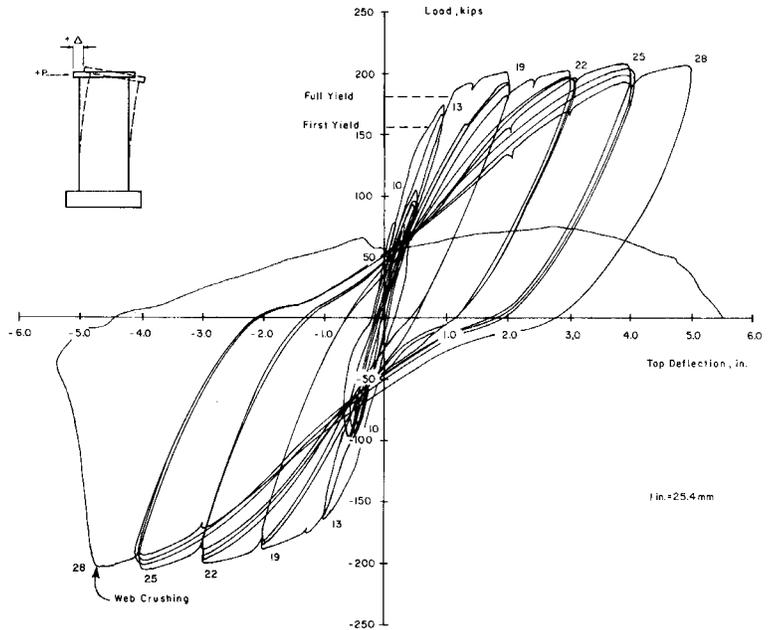
Load versus deflection relationship

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**		
Component Type:	**Isolated Wall or Stronger Wall Pier**	**RC1C**	**Example 2 of 3**
Predominant Behavior Mode:	**Flexure/Web Crushing**		
Secondary Behavior Mode:	**—**		

Reference: Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)
Specimen: B5
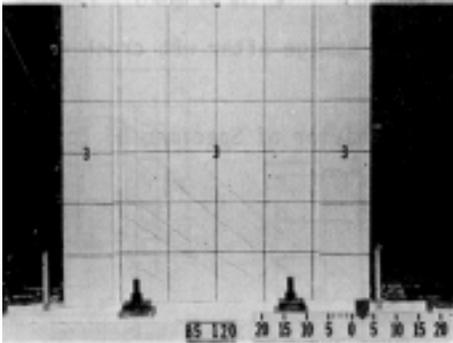
Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
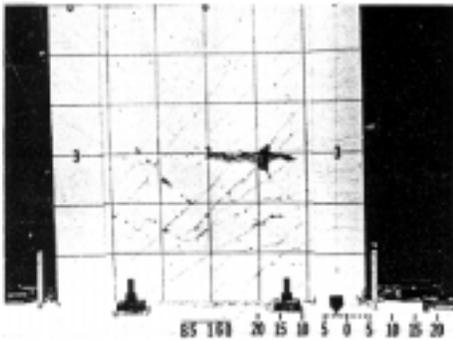
Damage at -3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
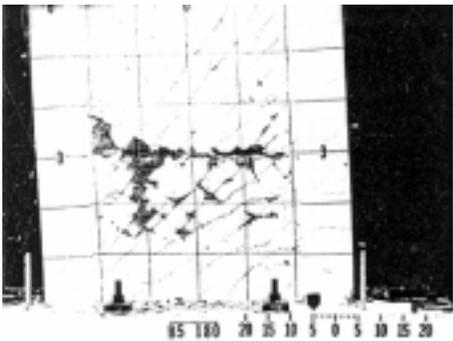
Damage after web crushing
$\Delta = 5$ in $\Delta/h_w = 0.028$ $\lambda_Q = 0.6$
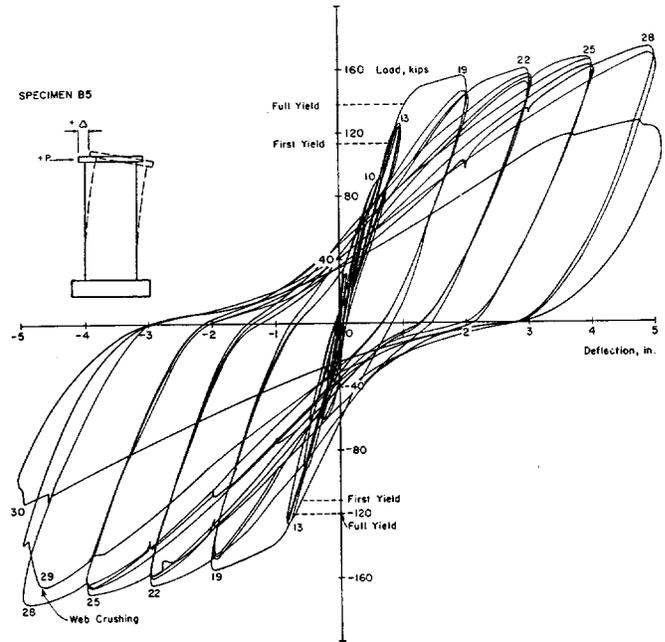
Load versus deflection relationship

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Web Crushing**
Secondary Behavior Mode:	**—**

RC1C   **Example 3 of 3**

Reference: Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)
Specimen: B6
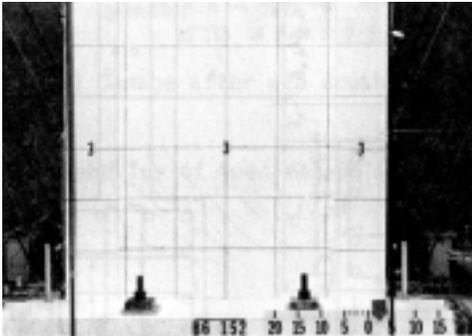
Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
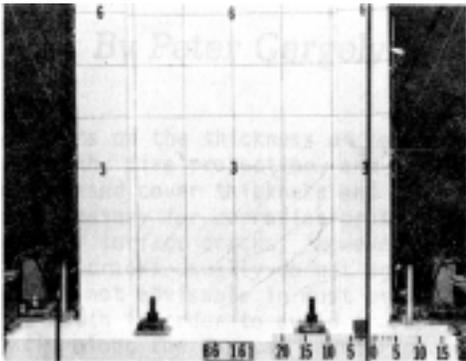
Damage at -3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
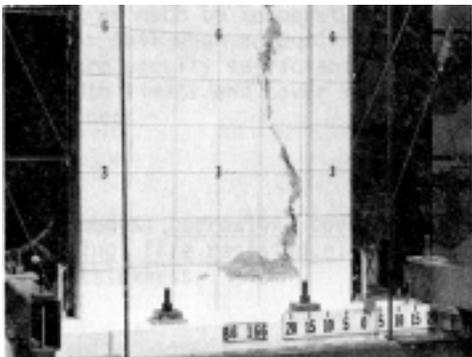
Damage after web crushing
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 0.3$
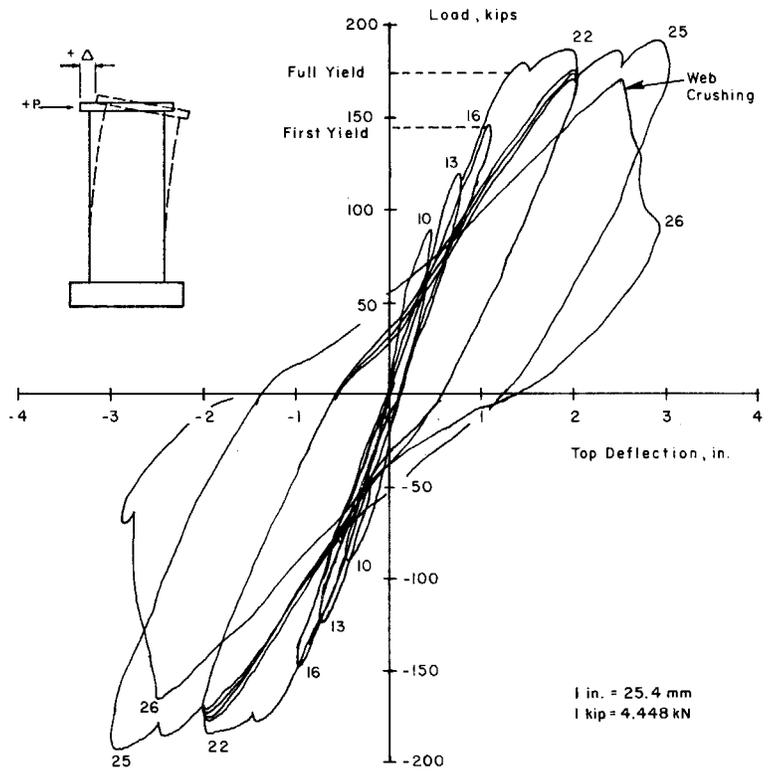
Load versus deflection relationship

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Sliding Shear**
Secondary Behavior Mode:	**—**

| RC1D | Example 1 of 2 |

Reference:	Corley, Fioralo, Oesterle (1981), Shiu et al. (1981)
Specimen:	CI-1
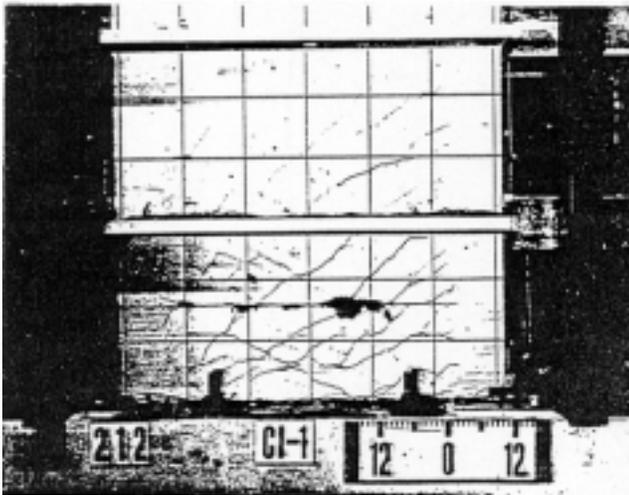
Crack pattern of specimen CI-1 at end of phase II.
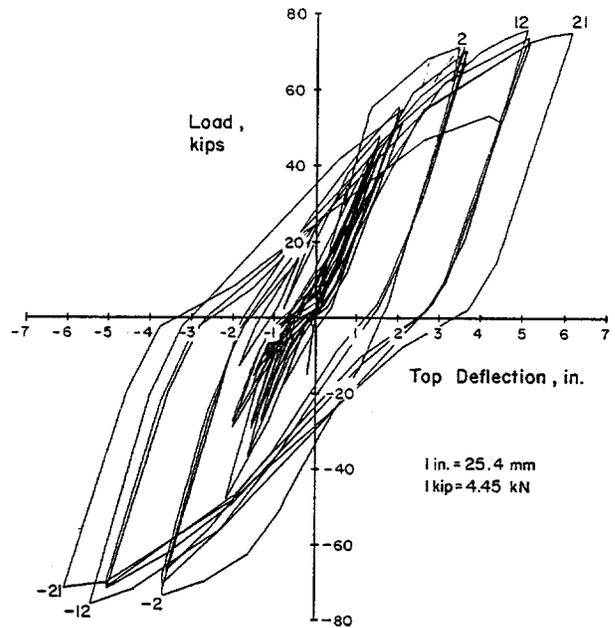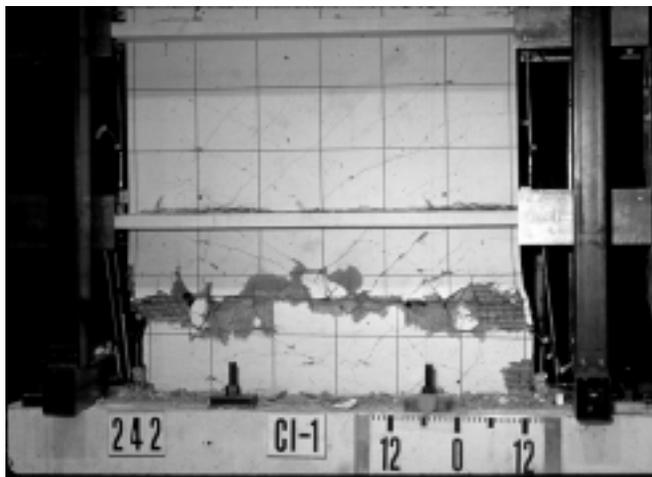
Load versus top deflection relationship for specimen CI-1.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Sliding Shear**
Secondary Behavior Mode:	**—**

RC1D   **Example 2 of 3**

Reference:	Paulay, Priestley, and Synge (1982))
Specimen:	Wall 1
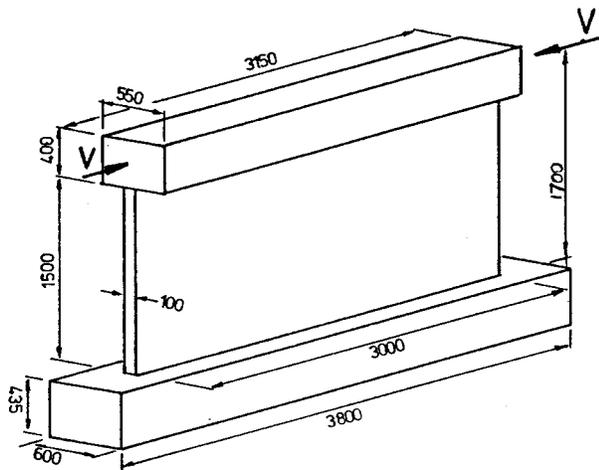
Overall dimensions of typical test units.

Splitting and Crushing of Concrete at Base of Wall
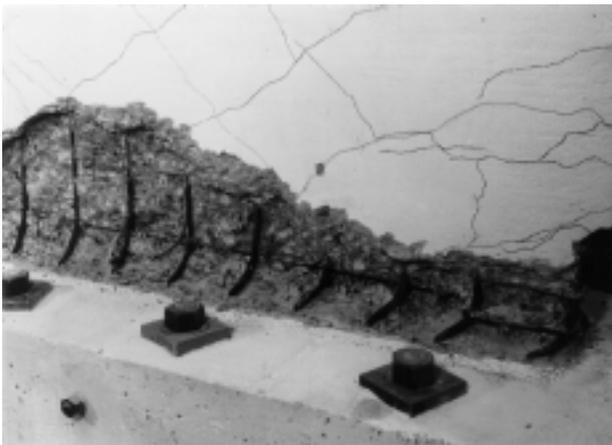
Compression Toe
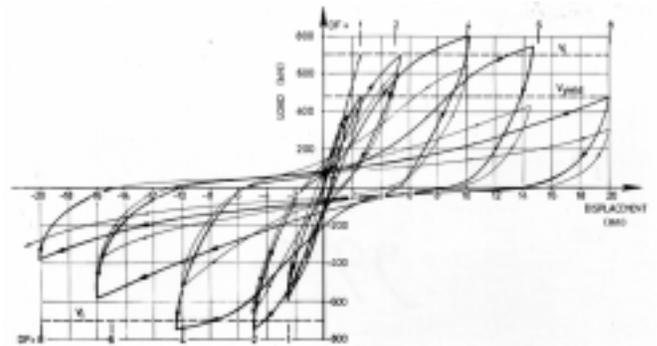
Load-deflection relationship for wall 1.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**		
Component Type:	**Isolated Wall or Stronger Wall Pier**	RC1D	**Example 3 of 3**
Predominant Behavior Mode:	**Flexure/Sliding Shear**		
Secondary Behavior Mode:	**—**		
Reference:	Paulay, Priestley, and Synge (1982))		
Specimen:	Wall 3		

Overall Dimensions for Walls 3 and 4.
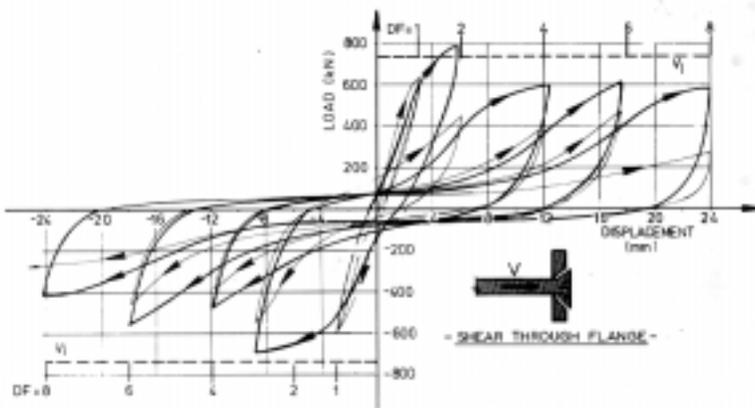
Load-Deflection Relationship for Flanged Wall

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Boundry Compression**
Secondary Behavior Mode:	**—**

RC1E   **Example 1 of 1**

Reference: Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)
Specimen: B1
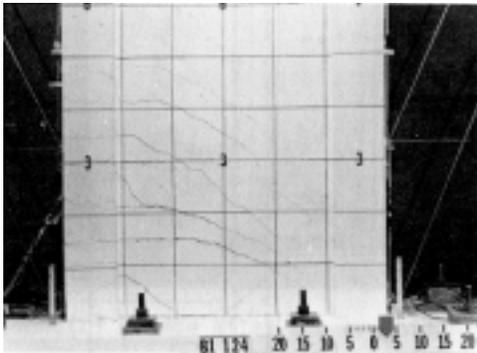
Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$
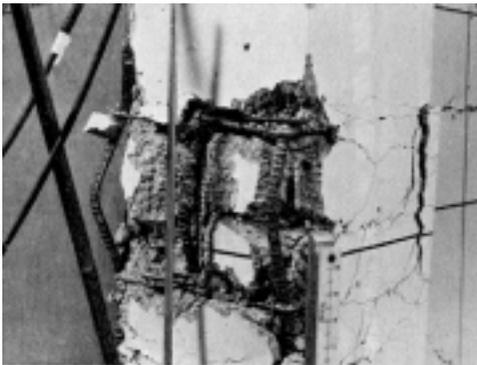
Buckled reinforcement after Load Cycle 30
$\Delta = 4$ in $\Delta/h_w = 0.022$ $\lambda_Q = 0.9$
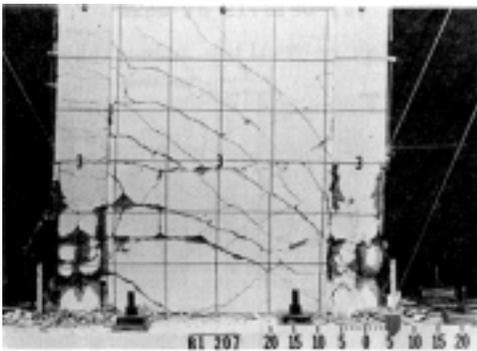
Damage during Load Cycle 34
$\Delta = 6$ in $\Delta/h_w = 0.033$ $\lambda_Q = 0.6$

Load versus deflection relationship

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Out-of-Plane Wall Buckling**
Secondary Behavior Mode:	**—**

$\boxed{\text{RC1G}}$ **Example 1 of 2**

Reference:	Corley, Fioralo, Oesterle (1981), Oesterle et al. (1976), Oesterle et al. (1979)
Specimen:	R2

Cracking pattern at +3 in. deflection for Specimen R2

Cracking pattern at -3 in. deflection for Specimen R2
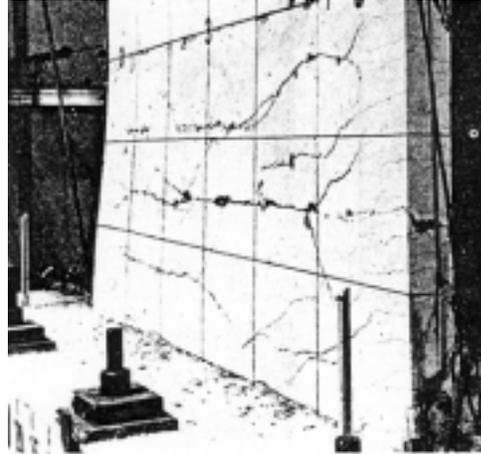
Inelastic instability of compression zone
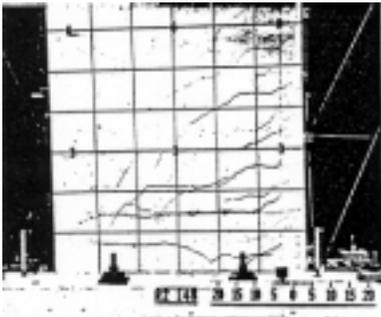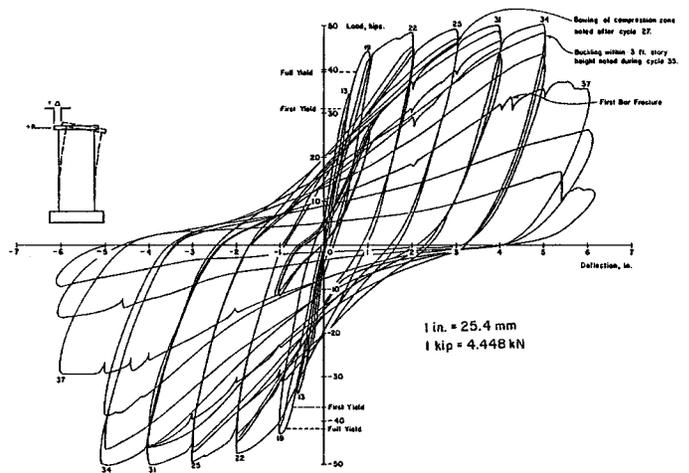
Continuous load-deflection plot for Specimen R2

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Flexure/Out-of-Plane Wall Buckling**
Secondary Behavior Mode:	**—**

$\boxed{\text{RC1G}}$ **Example 2 of 2**

Reference:	Paulay and Priestley (1992)
Specimen:	Wall 2 and Wall 4, Figure 5.37 of reference
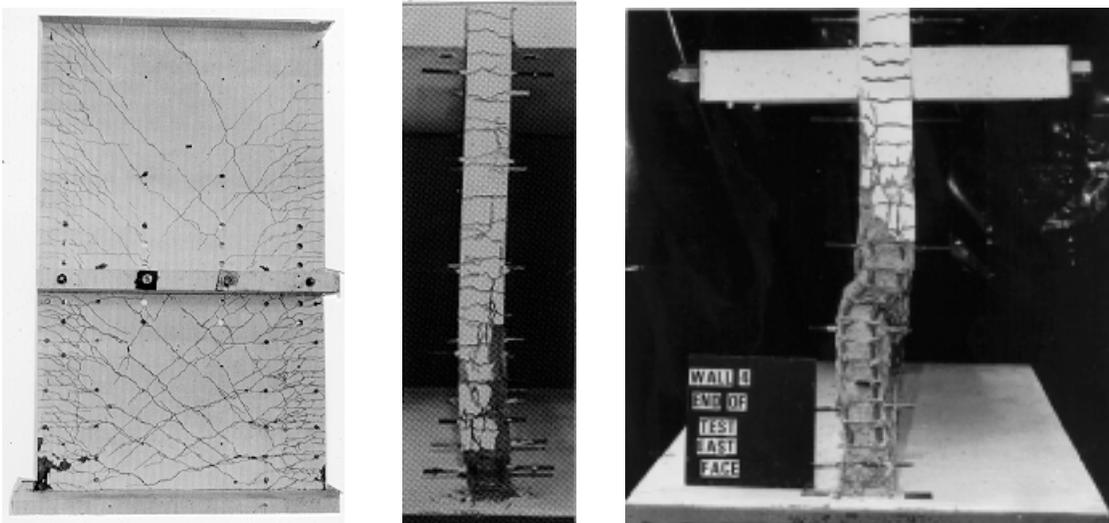
Diagonal cracking and buckling in the plastic hinge region of a structural wall (G1).
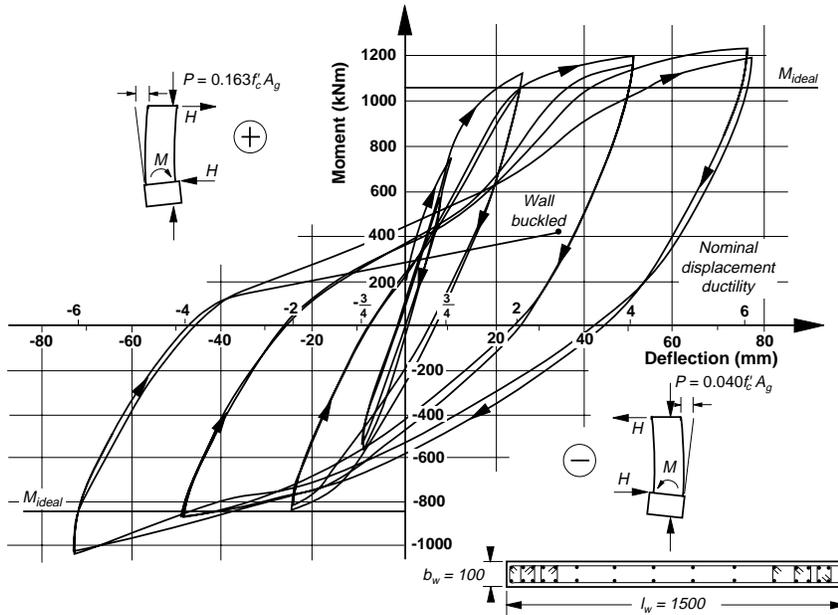
Stable hysteretic response of a ductile wall structure (G1).

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Preemptive Web Crushing**
Secondary Behavior Mode:	**—**

RC1I	**Example 1 of 2**

Reference:	Barda (1972), Barda, Hanson, & Corley (1976) (Lehigh Univ.)
Specimen:	B3-2

Test specimen at ultimate load
$\Delta = 0.2$ in $\Delta/h_w = 0.005$ $\lambda_Q = 1.0$
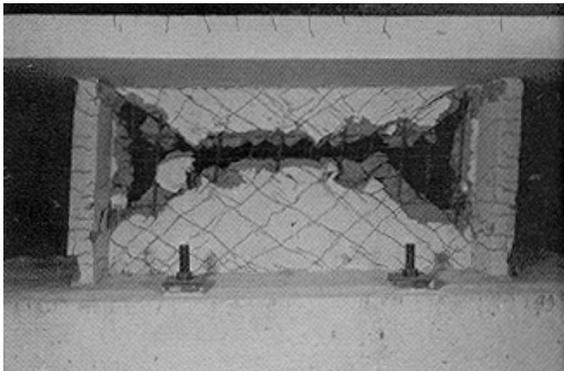
Test specimen at conclusion of loading
$\Delta = 3.0$ in $\Delta/h_w = 0.080$ $\lambda_Q = 0.2$

Envelope of response

Note: Shape in this region affected by loading procedure
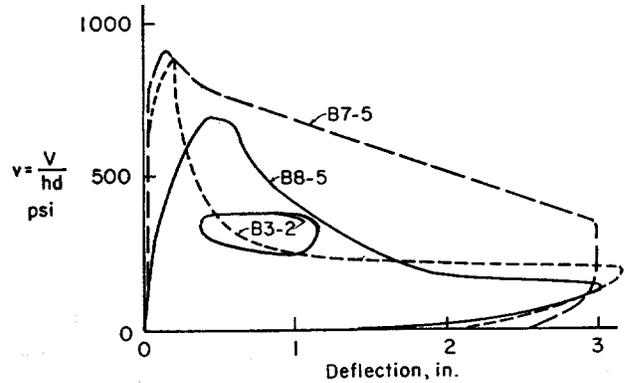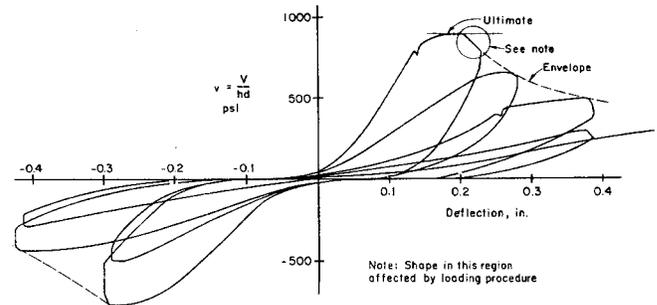
Hysteretic response
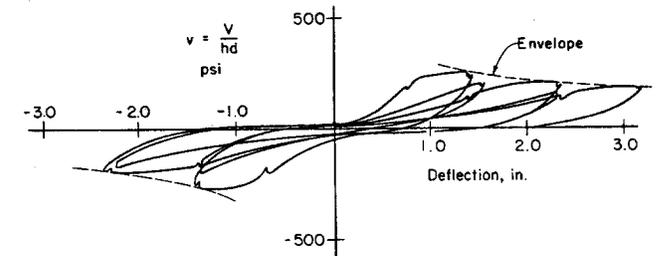
Provided Information **Calculated Values**

$h_w = 37.5$ "	$P = 4.9$ k
$f_y = 60$ ksi	$M_n = 1700$ k-1
$f'_c = 3920$ psi	$\dfrac{V}{b_w l_w}$ corresponding to $M_n = 1810$ psi

Δ	Δ/h_w	λ_Q
0.20	0.005	1.0
0.23	0.006	0.9
0.28	0.007	0.7
0.40	0.011	0.5
0.80	0.021	0.3
3.00	0.080	0.2

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**		
Component Type:	**Isolated Wall or Stronger Wall Pier**	**RC1I**	**Example 2 of 2**
Predominant Behavior Mode:	**Preemptive Web Crushing**		
Secondary Behavior Mode:	—		

Reference:	Barda (1972), Barda, Hanson, & Corley (1976)
Specimen:	B8-5

Test specimen at ultimate load
$\Delta = 0.2$ in $\Delta/h_w = 0.005$ $\lambda_Q = 1.0$

Envelope of response

$$v = \frac{V}{hd}\ \text{psi}$$
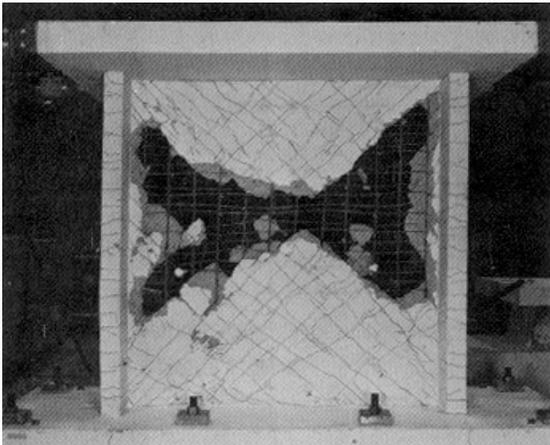
Test specimen at conclusion of loading
$\Delta = 3.0$ in $\Delta/h_w = 0.040$ $\lambda_Q = 0.2$

Provided Information	**Calculated Values**	Δ	Δ/h_w	λ_Q
$h_w = 75$ "	$P = 7.5$ k	0.45	0.006	1.0
		0.60	0.008	0.9
$f_y = 71$ ksi	$M_n = 2000$ k-1	0.80	0.011	0.7
$f_c' = 3400$ psi	$\dfrac{V}{b_w l_w}$ corresponding to $M_n = 1070$ psi	1.20	0.016	0.5
		1.70	0.023	0.3
		3.00	0.040	0.2

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Isolated Wall or Stronger Wall Pier**
Predominant Behavior Mode:	**Preemptive Sliding Shear**
Secondary Behavior Mode:	**Web Crushing**

$\boxed{\text{RC1J}}$ **Example 1 of 1**

Reference: Barda (1972), Barda, Hanson, & Corley (1976) (Lehigh Univ.)
Specimen: B7-5

Envelope of response
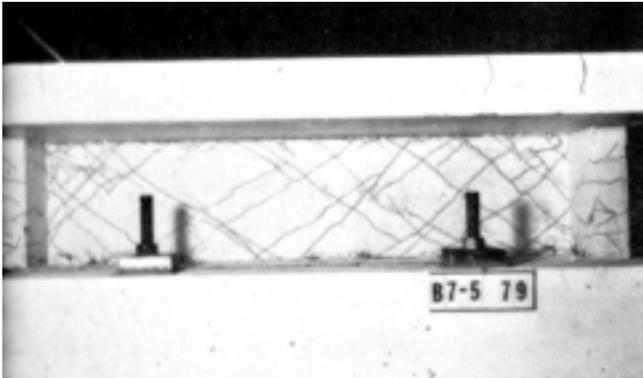
Test specimen at ultimate load
$\Delta = 0.15$ in $\Delta/h_w = 0.008$ $\lambda_Q = 1.0$

Hysteretic response to 0.6in.
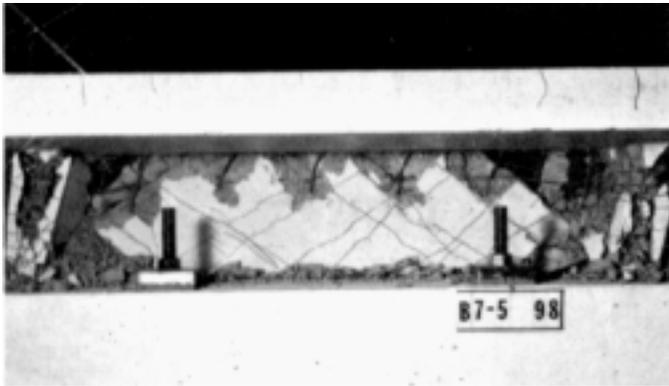
Test specimen at conclusion of loading
$\Delta = 3.0$ in $\Delta/h_w = 0.160$ $\lambda_Q = 0.4$

Provided Information

$h_w = 18.75$ "

$f_y = 78$ ksi

$f_c' = 3730$ psi

Calculated Values

$P = 3.6$ k

$M_n = 2180$ k-1

$\dfrac{V}{b_w l_w}$ corresponding to

$M_n = 4600$ psi

λ_Q **values from response plot**

Δ	Δ/h_w	λ_Q
0.15	0.008	1.0
0.30	0.016	0.9
0.70	0.037	0.8
1.80	0.096	0.6
3.00	0.160	0.4
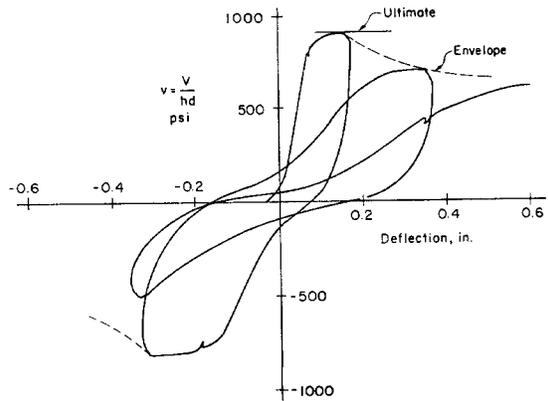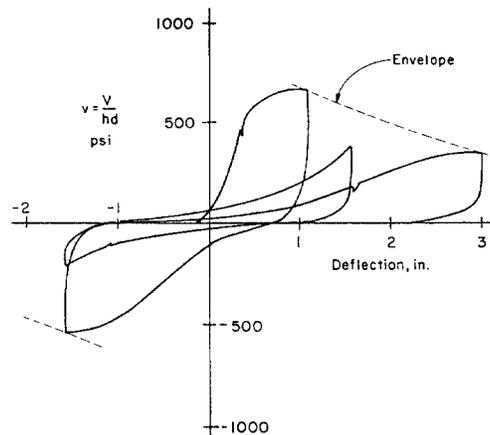
Hysteretic response to 3.0 in.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Weaker Spandrel or Coupling Beam**
Predominant Behavior Mode:	**Ductile Flexure**
Secondary Behavior Mode:	**—**

RC3A **Example 1 of 1**

Reference: Paulay & Binney (1974)
Specimen: Beam 316
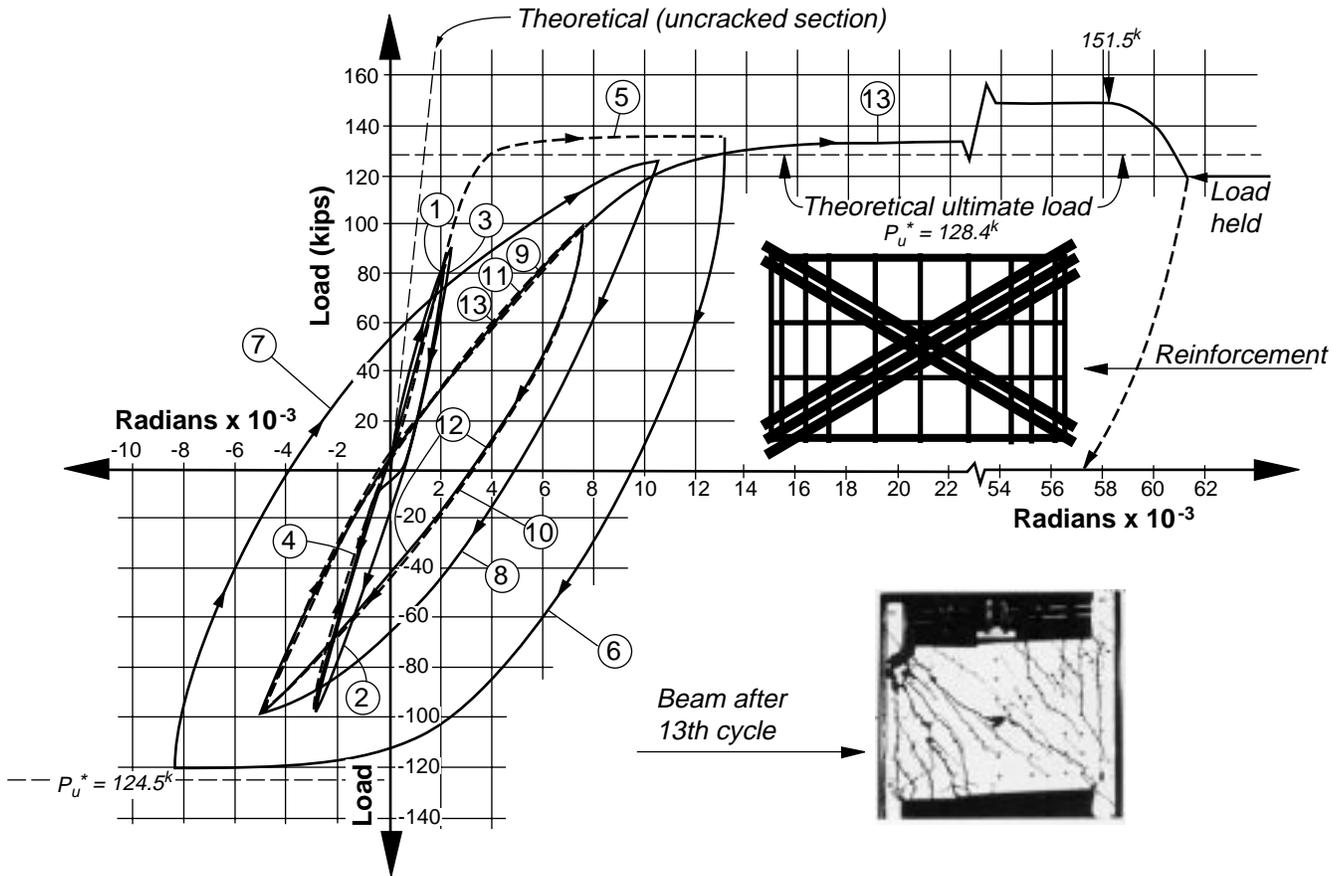
Load-rotation relationship for Beam 316.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**	
Component Type:	**Weaker Spandrel or Coupling Beam**	RC3D **Example 1 of 1**
Predominant Behavior Mode:	**Flexure/Sliding Shear**	
Secondary Behavior Mode:	**—**	

Reference:	Paulay & Binney (1974)
Specimen:	Beam 315

Load-rotation relationship for a conventional coupling beam.

DAMAGE PATTERNS AND HYSTERETIC RESPONSE

System:	**Reinforced Concrete**
Component Type:	**Weaker Spandrel or Coupling Beam**
Predominant Behavior Mode:	**Preemptive Diagonal Tension**
Secondary Behavior Mode:	**—**

RC3H   **Example 1 of 1**

Reference: Paulay (1977), Paulay (1986)
Specimen: Beam 392
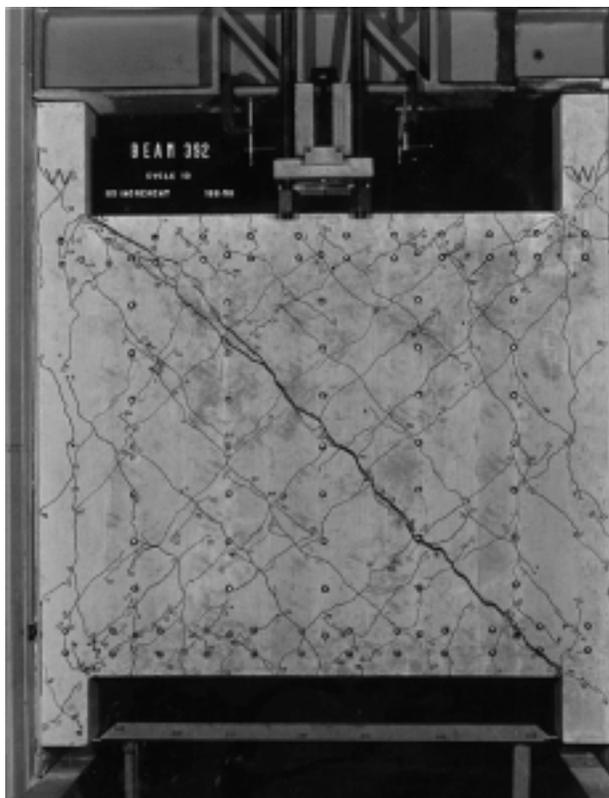
Beam 392 after being subjected to seismic-type loading: Cycle 13.
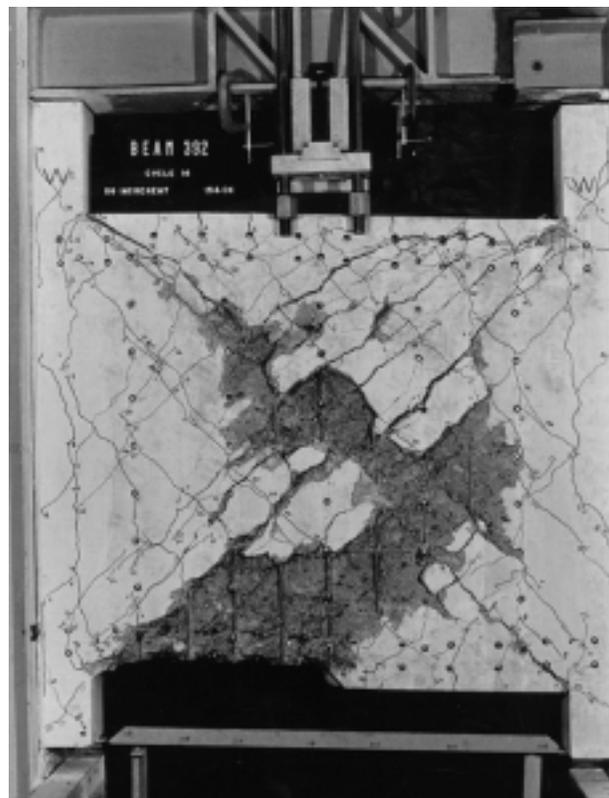
Beam 392, Cycle 14.

2.3   Tabular Bibliography

Table 2-2 contains a brief description of the key techni-
cal reports that address specific reinforced concrete
component behavior. The component types and their

behavior modes are indicated The full references can be
found in Section 2.5.

Table 2-2   Key References on Reinforced Concrete Wall Behavior.

Reference	Description	Comp. Types	Behavior modes Addressed											
			A	B	C	D	E	F	G	H	I	J	K	L
EVALUATION AND DESIGN RECOMMENDATIONS:														
ACI 318 (1995)	Code provisions for the design of r/c walls. Distinct behavior modes are often not considered explicitly.	RC1 – RC4												
Paulay & Priestley (1992)	Comprehensive recommendations for the design of r/c walls. Considers all component types and prevalent behavior modes.	RC1 – RC4	•	•	•	•	•		•	•			•	
Oesterle et al (1983)	Development of a design equation for web crushing strength. Strength is related to story drift and correlation with research results is shown.	RC1			•									
OVERVIEWS OF TEST RESULTS:														
Wood (1991)	Review of 27 specimens. 24 cyclic-static loading, 3 monotonic loading. "Slender" walls: $1.1 < M/VL < 2.9$, All specimens reached flexural yield. Failure categorized as either "shear" or "flexure".	RC1												
Wood (1990)	Review of 143 specimens. 50 cyclic-static loading, 89 monotonic loading, 4 repeated unidirectional loading. "Short" walls: $0.23 < M/VL < 1.7$. Review focuses on maximum strength. Failure modes and displacement capacity not addressed	RC1												
ATC-11(1983)	Commentary on implications of r/c wall test results and design issues.	RC1, RC3												
Sozen & Moehle (1993)	Review of wall test results applicable to nuclear power plant structures. Focused on predicting initial stiffness.	RC1												

[1] Behavior modes:

A Ductile Flexural Response

B Flexure/Diagonal Tension

C Flexure/Diagonal Compression (Web Crushing)

D Flexure/Sliding Shear

E Flexure/Boundary-Zone Compression

F Flexure/Lap-Splice Slip

G Flexure/Out-of-Plane Wall Buckling

H Preemptive Diagonal Tension

I Preemptive Web Crushing

J Preemptive Sliding Shear

K Preemptive Boundary Zone Compression Failure

L Preemptive Lap-Splice Failure

M Global foundation rocking of wall

N Foundation rocking of individual piers

Table 2-1   Key References on Reinforced Concrete Wall Behavior (continued)

Reference	Description	Comp. Types	Behavior modes Addressed											
			A	B	C	D	E	F	G	H	I	J	K	L
DETAILED TEST RESULTS:														
Barda (1972) Barda, Hanson & Corley (1976) (Lehigh Univ.)	8 test specimens: 6 cyclic-static loading, 2 monotonic loading, Small axial load. Approx. 1/3 scale, flanged walls. Low-rise: M/VL = 1.0, 0.5, 0.25. Wall vertical & horiz. reinf. and flange longit. reinf. varied 1 specimen repaired by replacement of web concrete and tested.	RC1									•	•		
Oesterle et al (1976) Oesterle et al (1979) (Portland Cement Association)	16 test specimens: 2 rectangular, 12 barbell, 2 flanged. M/VL = 2.4. Approx. 1/3 scale. Variables include boundary longit. and hoop reinf., wall horiz. reinf., axial load, load history 2 specimens repaired and tested.	RC1	•		•		•		•					
Shiu et al (1981) (Portland Cement Association)	2 test specimens. One solid wall and one wall with openings. Approx. 1/3 scale. Rectangular sections. Solid wall governed by sliding shear. Wall with openings was governed by diagonal compression in the piers. Coupling beams were not significantly damaged.	RC1, RC2, RC4			•	•								
Wang, Bertero & Popov (1975) Vallenas, Bertero & Popov (1979) (U.C. Berkeley)	10 test specimens: 6 barbell and 4 rectangular. 5 cyclic-static loading, 5 monotonic. 1/3 scale, modeled bottom 3 stories of 10-story barbell wall and 7-story rectangular wall. 5 specimens repaired with replacement of damaged rebar and crushed concrete.	RC1			•		•	•	•					
Iliya & Bertero (1980) (U.C. Berkeley)	2 test specimens. Barbell-shaped sections. Combination of cyclic-static and monotonic loading. 1/3 scale, modeled bottom 3 stories of 10-story barbell wall. Specimens repaired with epoxy injection of cracks after minor damage then subsequently repaired (after major damage) with replacement of damaged rebar and crushed concrete.	RC1			•			•						

[1] Behavior modes:

A Ductile Flexural Response

B Flexure/Diagonal Tension

C Flexure/Diagonal Compression (Web Crushing)

D Flexure/Sliding Shear

E Flexure/Boundary-Zone Compression

F Flexure/Lap-Splice Slip

G Flexure/Out-of-Plane Wall Buckling

H Preemptive Diagonal Tension

I Preemptive Web Crushing

J   Preemptive Sliding Shear

K Preemptive Boundary Zone Compression Failure

L Preemptive Lap-Splice Failure

M Global foundation rocking of wall

N Foundation rocking of individual piers

Table 2-1   Key References on Reinforced Concrete Wall Behavior (continued)

Reference	Description	Comp. Types	Behavior modes Addressed											
			A	B	C	D	E	F	G	H	I	J	K	L
Paulay, Priestley & Synge (1982)	4 test specimens, 2 rectangular, 2 flanged. Low-rise walls, $M/VL = 0.57$ Approx. 1/2 scale. Two specimens with diagonal bars to prevent sliding shear.	RC1	•			•								
Paulay & Binney (1974) Paulay (1971a, 1971b)	12 coupling-beam test specimens, 3 monotonic loading, 9 cyclic-static loading. $M/VL = 0.51, 0.65$. Approx. 1/2 scale. Varied amount of stirrup reinforcement, and amount and arrangement of longitudinal reinf., 3 specimens with diagonal bars.	RC3	•	•	•	•								
Paulay and Santhaku-mar (1976)	Two 7-story coupled wall specimens. Cyclic-static loading 1/4 scale. One specimen with diagonally reinforced coupling beams.	RC1 RC3	•			•								
Barney et al (1978) (Portland Cement Association)	8 coupling beam test specimens, Cyclic-static loading. $M/VL = 1.25, 2.5$. Approximately 1/3-scale specimens with conventional longitudinal reinforcement, diagonal bars in hinge zones, and full length diagonal bars. Full length diagonal reinforcement significantly improved performance.	RC3	•	•		•								
Wight (Editor) (1985)	7-story building, two bays by three bays with beam and slab floors, cyclic-static loading full scale. One wall acting parallel to moment frames. Parallel and perpendicular frames increased the capacity of the structure. Test structure repaired with epoxy injection and re-tested	RC1			•									
Alexander, Heide-brcht, and Tso (1973) (McMaster University)	$M/VL = 2.0, 1.33, 0.67$ Cyclic-static loading. 1/2 scale. Axial load varied.	RC1	•			•								
Shiga, Shibata, and Takahashi (1973 ,1975) (Tohoku University)	8 test specimens, 6 cyclic-static loading, 2 monotonic. Approx. 1/4 scale. Barbell section. Load history, web reinforcement, and axial load varied. $M/VL = 0.63$.	RC1			•									
Maier (1991)	10 test specimens, 2 cyclic-static loading, 8 monotonic. 7 flanged sections, 3 rectangular. Approx. 1/3 scale. Reinforcement and axial load varied. $M/VL = 1.12$.	RC1		•	•									

[1] Behavior modes:

A Ductile Flexural Response

B Flexure/Diagonal Tension

C Flexure/Diagonal Compression (Web Crushing)

D Flexure/Sliding Shear

E Flexure/Boundary-Zone Compression

F Flexure/Lap-Splice Slip

G Flexure/Out-of-Plane Wall Buckling

H Preemptive Diagonal Tension

I Preemptive Web Crushing

J   Preemptive Sliding Shear

K Preemptive Boundary Zone Compression Failure

L Preemptive Lap-Splice Failure

M Global foundation rocking of wall

N Foundation rocking of individual piers

Table 2-1   **Key References on Reinforced Concrete Wall Behavior (continued)**

Reference	Description	Comp. Types	Behavior modes Addressed											
			A	B	C	D	E	F	G	H	I	J	K	L
Mansur, Balendra, and H'ng (1991)	4 successful test specimens, cyclic-static loading. Approx. 1/4 scale. Flanged section. Web reinforced with welded wire mesh or expanded metal. M/VL = 0.68.	RC1		•	•									
Saatcioglu (1991)	3 test specimens, cyclic-static loading Approx. 1/3 scale. Rectangular section. Horizontal and sliding-shear dowel reinforcement varied. M/VL = 0.50.	RC1		•	•	•								
Aristizabal-Ochoa, Dario, & Sozen (1976) (University of Illinois)	4 shake-table specimens. Approx. 1/12 scale. 10-story coupled walls, rectangular pier and beam sections. Discusses reduced stiffness of coupling beams resulting from bond slip, and redistribution of demands between wall piers.	RC1 RC3	•											
Lybas & Sozen (1977) (University of Illinois)	6 test specimens, 5 shake-table and 1 cyclic static. Approx. 1/12 scale. 6-story coupled walls, rectangular pier and beam sections.	RC1 RC3	•											
Azizinamini et al. (1994) (Portland Cement Association)	Out-of-plane tests on tilt-up walls. 6 test specimens. Approx. 3/5 scale. Monotonic out-of-plane loading. Report shows typical crack patterns resulting from out-of-plane forces.	RC1												
ACI-SEAOSC Task Force (1982)	Out-of-plane tests on tilt-up walls, 12 reinforced concrete specimens (Also, 18 reinforced masonry specimens). Full scale monotonic out-of-plane loading and constant axial loading h/t ratios of 30 to 60. .	RC1												

[1] Behavior modes:

A Ductile Flexural Response

B Flexure/Diagonal Tension

C Flexure/Diagonal Compression (Web Crushing)

D Flexure/Sliding Shear

E Flexure/Boundary-Zone Compression

F Flexure/Lap-Splice Slip

G Flexure/Out-of-Plane Wall Buckling

H Preemptive Diagonal Tension

I Preemptive Web Crushing

J   Preemptive Sliding Shear

K Preemptive Boundary Zone Compression Failure

L Preemptive Lap-Splice Failure

M Global foundation rocking of wall

N Foundation rocking of individual piers

2.4 Symbols for Reinforced Concrete

Symbols that are used in this chapter are defined below. Further information on some of the variables used (particularly those noted "per ACI") may be found by looking up the symbol in Appendix D of *ACI 318-95*.

A_{ch} = Cross sectional area of confined core of wall boundary region, measured out-to-out of confining reinforcement and contained within a length *c'* from the end of the wall, FEMA 306, Section A2.3.7

A_{cv} = Net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in^2 (per ACI)

A_g = Gross cross sectional area of wall boundary region, taken over a length *c'* from the end of the wall, FEMA 306, Section A2.3.7

A_{sh} = Total cross-sectional area of transverse reinforcement (including crossties) within spacing *s* and perpendicular to dimension h_c. (per ACI)

b = Width of compression face of member, in (per ACI)

b_w = Web width, in (per ACI)

c = Distance from extreme compressive fiber to neutral axis (per ACI)

c' = Length of wall section over which boundary ties are required, per FEMA 306, Section A2.3.7

d_b = Bar diameter (per ACI)

d_{bt} = Bar diameter of tie or loop

f_c' = Specified compressive strength of concrete, psi (per ACI)

f_y = Specified yield strength of nonprestressed reinforcement, psi. (per ACI)

f_{yh} = Specified yield strength of transverse reinforcement, psi (per ACI)

h_c = Cross sectional dimension of confined core of wall boundary region, measured out-to-out of confining reinforcement

h_d = Height over which horizontal reinforcement contributes to V_s per FEMA 306, Section A2.3.6.b

h_w = Height of wall or segment of wall considered (per ACI)

k_{rc} = Coefficient accounting the effect of ductility demand on V_c per FEMA 306, Section A2.3.6.b

l_p = Equivalent plastic hinge length, determined according to FEMA 306, Section A2.3.3.

l_u = Unsupported length considered for wall buckling, determined according to FEMA 306, Section A2.3.9

l_n = Beam clear span (per ACI)

l_w = Length of entire wall or segment of wall considered in direction of shear force (per ACI). (For isolated walls and wall piers equals horizontal length, for spandrels and coupling beams equals vertical dimension i.e., overall depth)

M_{cr} = Cracking moment (per ACI)

M_e = Expected moment strength at section, equal to nominal moment strength considering expected material strengths.

M_n = Nominal moment strength at section (per ACI)

M_u = Factored moment at section (per ACI)

M/V = Ratio of moment to shear at a section. When moment or shear results from gravity loads in addition to seismic forces, can be taken as M_u/V_u

N_u = Factored axial load normal to cross section occurring simultaneously with V_u; to be taken as positive for compression, negative for tension (per ACI)

s = Spacing of transverse reinforcement measured along the longitudinal axis of the structural member (per ACI)

s_1 = spacing of vertical reinforcement in wall (per ACI)

V_c = Nominal shear strength provided by concrete (per ACI)

V_n = Nominal shear strength (per ACI)

V_p = Nominal shear strength related to axial load per Section

V_s = Nominal shear strength provided by shear reinforcement (per ACI)

V_u = Factored shear force at section (per ACI)

V_{wc} = Web crushing shear strength per FEMA 306, Section A2.3.6.c

α = Coefficient accounting for wall aspect ratio effect on V_c per FEMA 306, Section A2.3.6.b

β = Coefficient accounting for longitudinal reinforcement effect on V_c per FEMA 306, Section A2.3.6.b

δ = Story drift ratio for a component, corresponding to the global target displacement, used in the computation of V_{wc}, FEMA 306, Section A2.3.6.c

μ = Coefficient of friction (per ACI)

μ_Δ = Displacement ductility demand for a component, used in FEMA 306, Section A2.3.4, as discussed in Section 6.4.2.4 of *FEMA-273*. Equal to the component deformation corresponding to the global target displacement, divided by the effective yield displacement of the component (which is defined in Section 6.4.1.2B of *FEMA-273*).

ρ_g = Ratio of total reinforcement area to cross-sectional area of wall.

ρ_l = Local reinforcement ratio in boundary region of wall according to FEMA 306, Section A2.3.7

ρ_n = Ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv} (per ACI). (For typical wall piers and isolated walls indicates amount of horizontal reinforcement.)

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3. Reinforced Masonry

3.1 Commentary and Discussion

Several topics that are relevant to the development of the reinforced masonry component guides are addressed in this chapter.

3.1.1 Typical Hysteretic Behavior

The behavior modes described for reinforced masonry in FEMA 306, Section A3.2 are based on experimental research and field observation of earthquake damaged masonry buildings. Typical damage patterns and hysteretic response representative of different components and behavior modes are presented in Table 3-1

3.1.2 Cracking and Damage Severity

Cracks in a structural wall can provide information about previous displacements and component response. Aspects of cracking that relate to component behavior include:

- The orientation of cracks

- The number (density) of cracks

- The spacing of cracks

- The width of individual cracks

- The relative size of crack widths

In reinforced masonry with a flexural behavior mode, flexural cracks generally form in the mortar bed joints. At the base of a tall cantilever wall, flexural cracks may propagate across the entire length of the wall. Following an earthquake, flexural cracks tend to close due to gravity loads, and they may be particularly hard to locate in mortar joints. They are generally associated with ductile response and the natural engagement of vertical reinforcement; as a result, they do not provide a good measure of damage. When such cracks are visible, they are only used to identify behavior modes, not to assess the severity of damage.

Diagonal cracks reflect associated shear stresses, but they may be a natural part of ductile flexural action. In fully-grouted hollow brick or block masonry, diagonal cracks typically propagate through the units with short deviations along the mortar joints. Stair-step diagonal cracks are rare, and would indicate partial grouting and low-strength mortar. In plastic-hinge zones undergoing flexural response, diagonal cracks propagate from the ends of flexural cracks. In shear-dominated panels, diagonal cracks are more independent of flexural cracks.

In a flexurally-controlled wall, diagonal cracks are well-distributed and of uniform, small width. In a wall undergoing the transition from flexural response to shear response, one or two diagonal cracks, typically at the center of the wall, will grow wider than the others, dominating the response and concentrating shear deformations in a small area. A poorly-detailed wall undergoing preemptive shear behavior may have very few cracks until a critical, single diagonal crack opens.

In the investigation of earthquake-damaged concrete and masonry wall structures, cracks are the most visible evidence of damage. Because cracks are a striking and easily observed indication of the effect of earthquakes on walls, there is a strong temptation to overemphasize the relationship between crack width and the associated decrease (if any) in the strength and deformation capacity of a wall. Hanson (1996), has made the case that crack width alone is a poor indicator of damage severity. In recognition of this, the Component Damage Classification Guides in FEMA 306 do not rely on crack width as the only description of damage—numerous indicators of damage severity in reinforced masonry walls are described, among which crack width is only one. Cracking patterns can provide a wealth of information about the performance of a structural wall, but the location, orientation, number, and distribution of the cracks must be considered as important as, if not more important than, the crack width.

With the understanding that crack width must be considered in the context of all of the other parameters that can affect the behavior mode and damage severity of a wall, a rational approach is required to understand the influence of crack width on damage. This section outlines the basis of crack width limits specified in the Component Damage Classification Guides.

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		
RM1 Flexure *See Guide RM1A*	Priestley and Elder 1982		
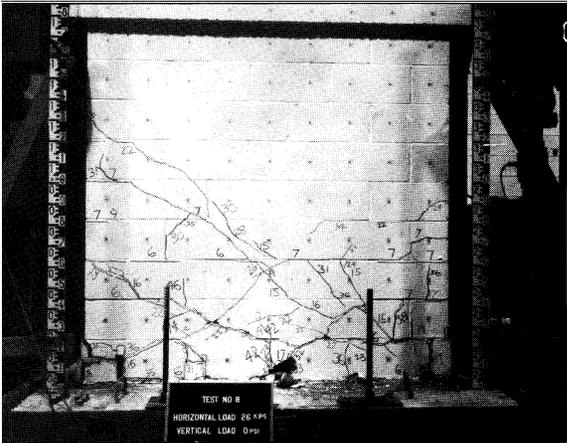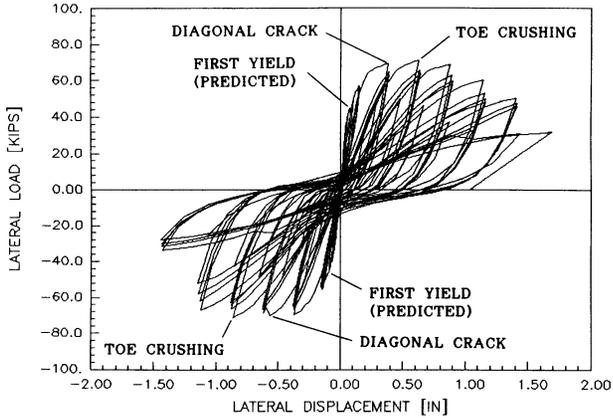
Table 3-1 **Damage Patterns and Hysteretic Response for Reinforced Masonry Components (continued)**
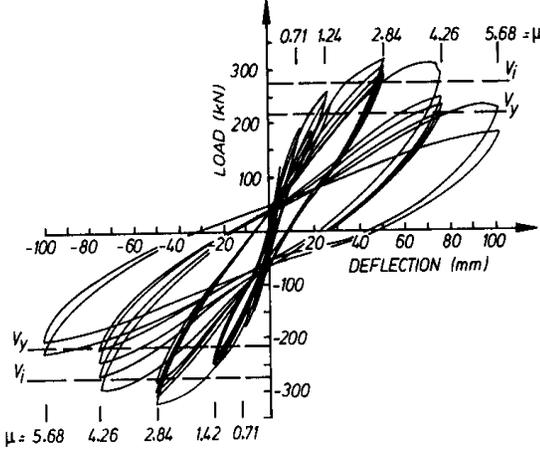
RM1 Flexure / Shear *See Guides RM1B and RM2B*	Shing et al., 1991 Specimen 7		
RM1 Flexure / Shear *See Guides RM1B and RM2B*	Priestley and Elder 1982		(a) Unconfined Wall
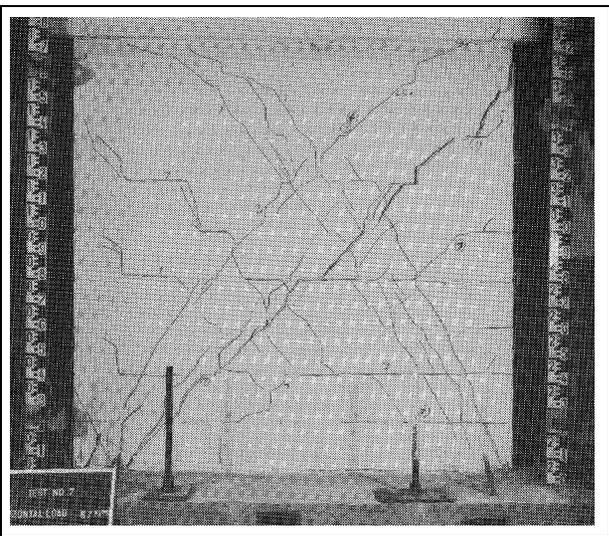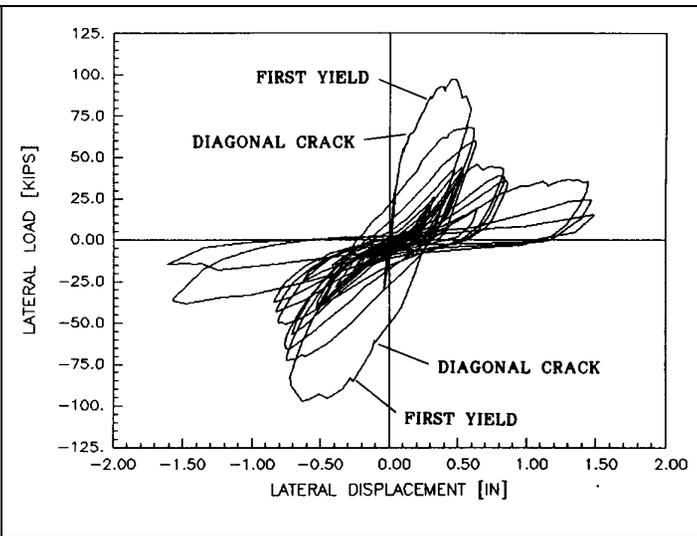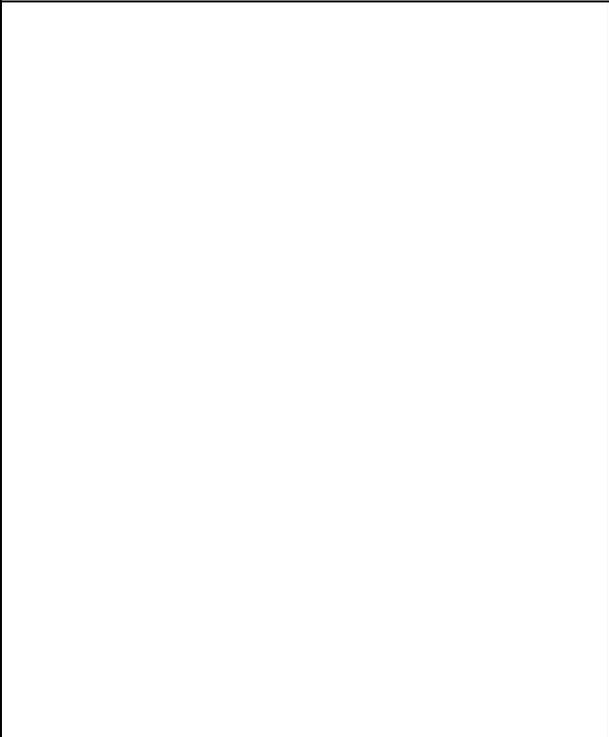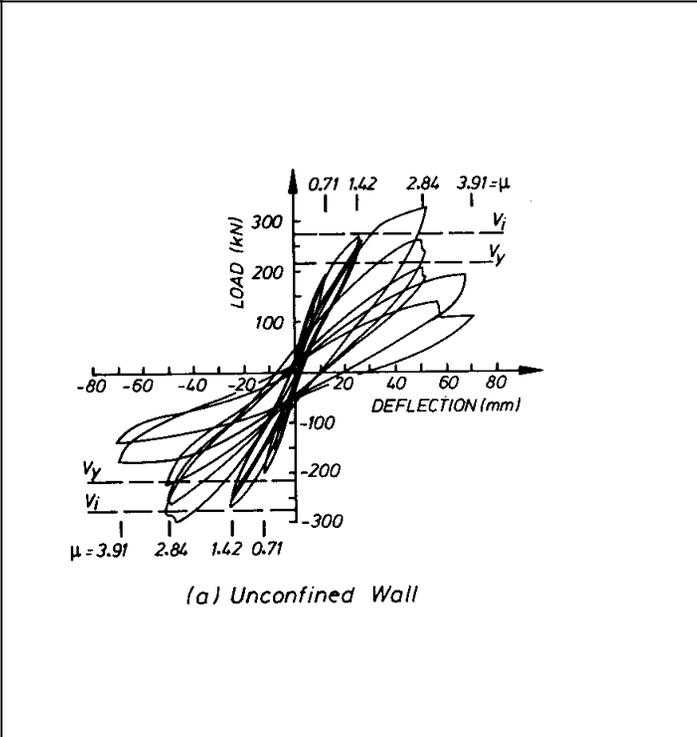
Table 3-1   Damage Patterns and Hysteretic Response for Reinforced Masonry Components (continued)

| RM1

Flexure / Sliding Shear

See Guide RM1C | Shing et al., 1991
Specimen 8 | | |
| RM1
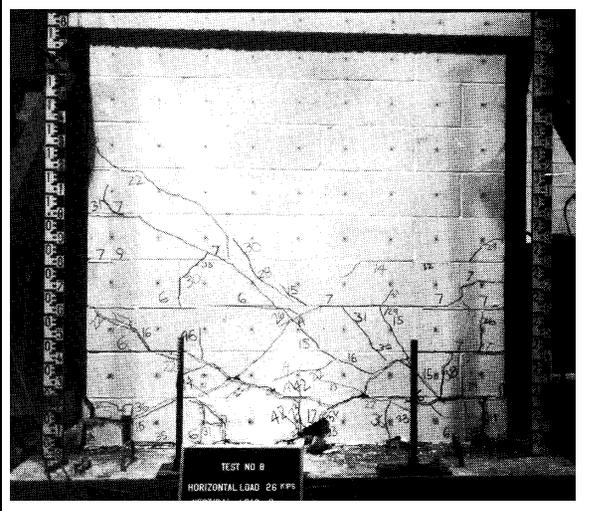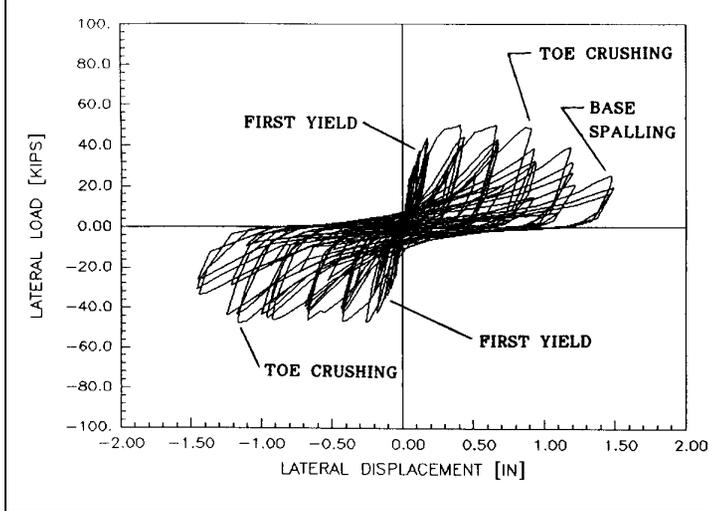
Flexure / Shear / Sliding Shear

See Guides RM1B and RM1C | Shing et al., 1991
Specimen 6 | | |

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

RM1 Flexure /lap splice slip *See Guide RM1E*	Shing et al., 1991 Specimen 19		TOE CRUSHING DIAGONAL CRACK FIRST YIELD LATERAL LOAD [KIPS] FIRST YIELD TOE CRUSHING LATERAL DISPLACEMENT [IN]
RM1 or RM2 Preemptive Shear *See Guide RM2G*	Shing et al., 1991 Specimen 9		DIAGONAL CRACK FIRST YIELD LATERAL LOAD [KIPS] FIRST YIELD DIAGONAL CRACK

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		
RM3 Flexure *See Guide RM3A*	Priestley and Hon, 1985		

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

RM1 or RM2 with flange. Flexure / Shear *See Guides RM1A, RM1B, and / or RM2G*	Priestley and He, 1990		

Research has been conducted to evaluate the relationship between crack width, crack spacing, and reinforcing bar strain. A partial review of the literature on crack width is provided by Noakowski, (1985). Research indicates that the width of a crack crossing a reinforcing bar at first yield of the reinforcement depends on the bar diameter, the reinforcement yield stress, the reinforcement ratio, the reinforcement elastic modulus, and on the characteristics of the bond stress-slip relationship. However, most research in this area has focused on nearly elastic systems (prior to yield in reinforcement), and flexural cracking in beams and uniaxial tension specimens. It is difficult to extrapolate quantitative expressions for crack width and spacing prior to yield to reinforced masonry specimens with sufficient damage to reduce strength or deformation capacity.

Sassi and Ranous (1996) have suggested criteria to relate crack width to damage, but they have not provided sufficient information to associate crack patterns with specific behavior modes, which is essential when determining damage severity.

In the guides for reinforced masonry components, the crack width limits for each damage severity level have been determined empirically, using crack widths reported in the literature and photographs of damaged specimens. Consideration has been given to the theoretical crack width required to achieve yield of reinforcement under a variety of conditions. A fundamental presumption is that the width of shear cracks is related to damage severity, while flexural crack widths are not closely related to damage severity.

3.1.3 Interpretation of Tests

Interpretation of test results for reinforced masonry was similar to that for reinforced concrete as described in Section 2.1.1.2. The ranges of component ductility and l-factors are presented in Table 3-2.

3.2 Tabular Bibliography for Reinforced Masonry

Table 3-3 contains a brief description of the key technical reports which address specific reinforced masonry component behavior. The component types and their behavior modes are indicated. The full references can be found in Section 3.4.

Table 3-2   *Ranges of reinforced masonry component displacement ductility, μ_Δ, associated with damage severity levels and λ factors*

Damage Guide	Damage Severity			
	Insignificant	**Slight**	**Moderate**	**Heavy**
RM1A Ductile Flexural	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2-4$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3-8$ $\lambda_K = 0.4$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Heavy* not used
RM1B Flexure/Shear	$\mu_\Delta \leq 2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2-3$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3-5$ $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	
RM1C Flexure/ Sliding Shear	See RM1A	$\mu_\Delta \approx 2-4$ $\lambda_K = 0.5$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3-8$ $\lambda_K = 0.2$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	
RM1D Flexure/ Out-of-Plane Instability	See RM1A	See RM1A	See RM1A	$\mu_\Delta \approx 8-10$ $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.5$
RM1E Flexure/ Lap Splice Slip	See RM1A or RM1B	See RM1A or RM1B	$\mu_\Delta \approx 3-4$ $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	
RM2B Flexure/Shear	$\mu_\Delta \leq 2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2-3$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3-5$ $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Heavy* not used
RM2G Preemptive Shear	$\mu_\Delta \leq 1$ $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 1-2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 1-2$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	$\mu_\Delta \approx 2-3$ $\lambda_K = 0.3$ $\lambda_Q = 0.4$ $\lambda_D = 0.5$
RM3A Flexure	$\mu_\Delta \leq 2$ $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	$\mu_\Delta \approx 6$ $\lambda_K = 0.6$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	
RM3G Preemptive Shear (No μ values for RM3G)	$\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\lambda_K = 0.8$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	$\lambda_K = 0.3$ $\lambda_Q = 0.5$ $\lambda_D = 0.9$	

Table 3-3   Annotated Bibliography for Reinforced Masonry

Reference(s)	Description			Comp. Type(s)	Behavior modes Addressed						
					a	b	c	d	e	f	g
EVALUATION AND DESIGN RECOMMENDATIONS											
Paulay and Priestley (1992)	Overview of capacity-design principles for reinforced concrete and masonry structures. Thorough description of R/C failure modes, and, to a lesser extent, R/M failure modes.	Description of R/M component response in terms of displacement and ductility.		RM1 RM2 RM3 RM4	•	•	•	•	•	•	•
OVERVIEWS OF EXPERIMENTAL TEST RESULTS											
Drysdale, Hamid, and Baker (1994)	Textbook for design of masonry structures. Includes complete bibliography and selected results from experimental research.			RM1 RM2 RM4	•	•					•
EXPERIMENTAL TEST RESULTS											
Abrams and Paulson (1989) Abrams and Paulson (1990)	<u>2 specimens</u> 1/4-scale model			RM2	•	•	•				
Foltz and Yancy (1993)	<u>10 Specimens</u> 8" CMU 56" tall by 48" wide Axial load 200+ psi	No vertical reinforcement $\rho_v = 0.0\%$ $\rho_h = 0.024\% - 0.22\%$ Axial load increased w/ displacement. Clear improvement in displacement and crack distribution w/ increased horizontal reinforcement.	Many damage photos. No hysteresis curves. Joint reinforcement improved ultimate displacement from $\mu=1$ to $\mu=3$.	RM2							•

Table 3-3   Annotated Bibliography for Reinforced Masonry (continued)

Reference	Specimens			Class							
Ghanem et al. (1993)	<u>14 Specimens</u> 1/3 scale concrete block	Monotonic tests only reported here.		RM2							•
Hammons et al. (1994)	<u>124 specimens</u> Hollow concrete and clay masonry	Monotonic testing of lap splices. Only #4 in 8" units fail by classical pull-out. Others fail in tensile splitting.	Tensile splitting failure likely regardless of lap splice length for: #4 in 4 inch units #6 in 6 inch units #8 in 8 inch units	N/A							
Hidalgo et al. (1978) Chen et al. (1978) Hidalgo et al. (1979)	<u>63 specimens:</u> 28 8" hollow clay brick 18 2-wythe clay brick 17 8" hollow concrete block	Aspect ratios: 0.5, 1.0, 2.0 High axial loads, increasing with lateral displacement.	All failures in shear or flexure/ shear	RM2	•						•
Hon & Priestley (1984) Priestley & Hon (1985) Hart & Priestley (1989) Priestley (1990)	<u>2 fully-grouted specimens</u> 8" hollow concrete block One specimen tested in New Zealand, and a second later at UC San Diego.	Full-scale, fully-reversed cyclic loading. 2nd specimen purposely violated proposed design criteria, and performed in a ductile manner.	Stable hysteresis up to displacement ductility of 4 at first crushing. Achieved ductility of 10 with minor load degradation.	RM3	•						
Igarashi et al. (1993)	<u>1 fully grouted 3-story wall specimen</u> 6" hollow concrete block 3-story full-scale cantilever wall	$\rho_v = 0.15\%$ $\rho_h = 0.22\%$	Flexural response to 0.3% drift followed by lap-splice slip at base and stable rocking to 1% drift at approx. 1/3 of max. load.	RM1				•			
Kubota and Murakami (1988)	<u>5 cmu wall specimens</u> Investigated effect of lap splices	Sudden loss of strength associated w/ lap-splice failure. Test stopped following lap-splice failure	Vertical splitting at lap	RM2	•			•			
Kubota et al. (1985)	<u>5 wall specimens</u> Hollow clay brick	Minimum vertical reinf $\rho_h = 0.17\% - 0.51\%$		RM2	•						•
Matsumura (1988)	Includes effect of grout flaws on damage patterns and shear strength.	Missing or insufficient grout causes localized damage and inhibits uniform distribution of cracks.		RM2	•						•

Table 3-3   Annotated Bibliography for Reinforced Masonry (continued)

Reference	Specimens	Design/Properties	Behavior	Type						
Matsuno et al. (1987)	<u>1 grouted hollow clay specimen</u> 3-stories 3-coupled flanged walls	Limited ductility, significant strength degradation associated w/ preemptive shear failure of coupling beams.	Flexure response in long wall (RM1) Flexure/shear in short walls (RM2)	RM1 RM2 RM4		•		•		•
Merryman et al (1990) Leiva and Klingner (1991)	<u>6 fully-grouted, 2-story wall specimens</u> 2-story walls with openings 2-story pairs of wall coupled by slab only 2-story pairs of walls coupled by slab and R/M lintel	Flexural design by 1985 UBC. Shear design to ensure flexure hinging. $\rho_v = 0.22\%$ $\rho_h = 0.22\% - 0.44\%$	Stable flexural response in coupled walls, limited by compression toe spalling, fracture of reinforcement, and sliding. No significant load degradation even at end of test. One specimen inadvertently loaded to 60% of max base shear in single pulse prior to test, with no clear effect on response.	RM1	•		•			
Okada and Kumazawa (1987)	Concrete block beams 32"x90"	Similar to concrete. Rotation capacity of 1/100	Damage for lap splices limited to splice zone. More distributed without laps.	RM4	•	•				
Priestley and Elder (1982)				RM1	•	•				
Schultz, (1996)	<u>6 partially-grouted specimens</u> concrete masonry	Minimum vertical reinf $\rho_h = .05\% - .12\%$ Moderately ductile response w/ initial peak and drop to degrading plateau at approx. 75% of max.	Drift = 0.3%-1% at 75% of max strength. Behavior characterized by vertical cracks at junction of grouted and ungrouted cells. Few if any diagonal cracks except in one specimen.	RM2		•				

Table 3-3   Annotated Bibliography for Reinforced Masonry (continued)

Reference	Description	Design/Loading	Behavior/Results	Type								
Seible et al. (1994) Seible et al. (1995) Kingsley (1994) Kingsley et al. (1994) Kürkchübasche et al. (1994)	<u>1 fully grouted, 5-story building specimen</u> 6" hollow concrete block 5-story full-scale flanged walls coupled by topped, precast plank floor system $\rho_v = 0.23\%$-0.34% $\rho_h = 0.11\%$ - 0.44%	Flexural design by 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings.* Shear design to ensure flexural hinging.	Ductile flexural response with some sliding to μ=6 and 9, (drift = 1% and 1.5%). Distributed cracking. Significant influence of flanges and coupling slabs.	RM1	•		•					
Shing et al. (1990a) Shing et al. (1990b) Shing et al. (1991)	<u>24 fully-grouted test specimens:</u> 6 6-inch hollow clay brick 18 6-inch hollow concrete block 2 monotonic loading 22 cyclic-static loading. 4 levels of axial load	Full-scale walls, 6-ft square, loaded in single curvature. $M/VL = 1$ Uniformly distributed vertical & horizontal reinforcement. $\rho_v = 0.38\%$ - 0.74% $\rho_h = 0.14\%$ - 0.26%	2 specimens with lap splices at base, others with continuous reinforcement. 1 specimen w/ confinement comb at wall toe. Most comprehensive tests on reinforced masonry wall components to date	RM1 RM2	•	•	•					•
Tomazevic and Zarnic (1985) Tomazevic and Lutman (1988) Tomazevic and Modena (1988) Tomazevic et al. (1993)	<u>32 wall specimens</u> Concrete block walls and complete structures Static and shaking table	$\rho_v = 0.26\%$ - 0.52% $\rho_h = 0.00\%$ - 0.52%		RM2	•	•						•
Yamazaki et al. (1988a) Yamazaki et al. (1988b)	<u>1 fully-grouted 5-story building specimen</u> 8" hollow concrete block 5-story full-scale flanged walls coupled by cast-in-place 6" and 8" R/C floor slabs	First damage in masonry lintel beams of many different geometries.	Flexural modes degraded to shear failing modes at 0.75% building drift (1.4% first story drift).	RM1 RM2 RM4		•						•

Table 3-3   Annotated Bibliography for Reinforced Masonry (continued)

EXPERIMENTAL TEST RESULTS – REPAIRED OR RETROFITTED WALLS

					a	b	c	d	e	f	g
Innamorato (1994)	3 fully-grouted test specimens Designed to match Shing (1991) Preemptive shear failure Flexure failure	Tested in "original" and "repaired" condition	Repair by epoxy injection and carbon fiber overlay	RM1 RM2	•	•					•
Laursen et al. (1995)	2 in-plane specimens Designed to match Shing (1991) specimen preemptive shear failure. 2 out-of-plane specimens	Tested in "original," "repaired," and "retrofit" configurations.	Repair by epoxy injection and carbon fiber overlays in horizontal or vertical direction to enhance ductility or strength	RM1 RM2	•						•
Weeks et al. (1994)	5-story building tested previously by Seible et al. (1994) repaired and retested.		Repair by epoxy injection and carbon fiber overlay	RM1							

[1] Behavior modes:

a Ductile Flexural Response:

b Flexure/Diagonal Shear

c Flexure/Sliding Shear

d Flexure/Out-of-Plane Wall Buckling

e Flexure/Lap-Splice Slip

f Foundation rocking of individual piers

g Preemptive Diagonal Shear Failure

3.3 Symbols for Reinforced Masonry

A_g = Gross crosssectional area of wall

A_{si} = Area of reinforcing bar i

A_v = Area of shear reinforcing bar

A_{vf} = Area of reinforcement crossing perpendicular to the sliding plane

a = Depth of the equivalent stress block

c = Depth to the neutral axis

C_m = Compression force in the masonry

f_{me} = Expected compressive strength of masonry

f_{ye} = Expected yield strength of reinforcement

h_e = Effective height of the wall (height to the resultant of the lateral force) = M/V

l_d = Lap splice development length

l_p = Effective plastic hinge length

l_w = Length of the wall

M/V = Ratio of moment to shear (shear span) at a section

M_e = Expected moment capacity of a masonry section

P_u = Wall axial load

s = Spacing of reinforcement

t = Wall thickness

V_e = Expected shear strength of a reinforced masonry wall

V_m = Portion of the expected shear strength of a wall attributed to masonry

V_s = Portion of the expected shear strength of a wall attributed to steel

V_p = Portion of the expected shear strength of a wall attributed to axial compression effects

V_{se} = Expected sliding shear strength of a masonry wall

x_i = Location of reinforcing bar i

Δ_p = Maximum inelastic displacement capacity

Δ_y = Displacement at first yield

ϕ_m = Maximum inelastic curvature of a masonry section

ϕ_y = Yield curvature of a masonry section

μ_Δ = Displacement ductility

μ = Coefficient of friction at the sliding plane

3.4   References for Reinforced Masonry

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4. Unreinforced Masonry

4.1 Commentary and Discussion

4.1.1 Hysteretic Behavior of URM Walls Subjected to In-Plane Demands

A search of the available literature was performed to identify experimental and analytical research relevant to unreinforced masonry bearing-wall damage. Because URM buildings have performed poorly in past earthquakes, there is an extensive amount of anecdotal information in earthquake reconnaissance reports; there have also been several studies that took a more statistical approach and collected damage information in a consistent format for a comprehensive population of buildings. These studies help to confirm the prevalence of the damage types listed in FEMA 306, and they help to indicate the intensity of shaking required to produce certain damage types.

The proposed methodology for this document, however, requires moving beyond anecdotal and qualitative discussions of component damage and instead obtaining quantitative information on force/displacement relationships for various components. The focus of research on URM buildings has been on the in-plane behavior of walls. Most of the relevant research has been done in China, the former Yugoslavia, Italy, and the United States. This stands in contrast to the elements in URM buildings that respond to ground shaking with essentially brittle or force-controlled behavior: parapets, appendages, wall-diaphragm ties, out-of-plane wall capacity, and, possibly, archaic diaphragms such as brick arch floors. While there has been very little research on most of these elements, it is less important because performance of these elements is not deformation-controlled.

Unfortunately, research on in-plane wall behavior is rarely consistent—materials, experimental techniques, modes of reporting, and identified inelastic mechanisms all vary widely. Placing the research in a format consistent with FEMA 273 and this project's emphasis on components, damage types, hysteresis curves, nonlinear force/displacement relationships, and performance levels is difficult. Almost no experimental tests have been done on damaged URM walls; typically, tests were done on undamaged walls and stopped. In some cases, the damaged wall was repaired and retested. Most of the research does not provide simple predictive equations for strength and stiffness (particularly post-elastic stiffness); when analysis has been done, it has usually used fairly sophisticated finite element modelling techniques.

Hysteresis loops for in-plane wall behavior are shown on the following pages, Sections 4.1.1.1 to 4.1.1.6, organized by behavior mode. Research shows that the governing behavior mode depends upon a number of variables including material properties, aspect ratio, and axial stress. To aid in comparing the curves, basic data given in the research report are provided, including the average compressive strength of prism tests and the masonry unit, the pier aspect ratio, the nominal axial stress, and whether the specimen was free to rotate at the top (cantilever condition) or was fixed (double-curvature condition). For many of the specimens, independent calculations have been carried out for this document to allow comparison between the evaluation procedure predictions in Section 7.3 of FEMA 306 and the actual experimental results. Predictions using FEMA 273 are also noted. In several cases, engineering judgment has been exercised to make these calculations, since not all of the necessary information is available. Material properties that were assumed for the purposes of the calculation are identified. It is expected that predicted results could vary significantly if different assumptions are made. In addition, the experimental research in URM piers is difficult to synthesize for several reasons:

- Some researchers do not report a measure of bed-joint sliding-shear strength. Others use triplet tests rather than in-place push tests to measure bed-joint sliding capacity. Comparisons between triplet tests and in-place push tests are not well established. Several different assumptions were investigated for this project, and the approach shown below was found to correlate best with the data.

- Descriptions of cracking can be inconsistent and overly vague. Diagonal cracking, for example, is often reported, but it can be unclear if the report refers to diagonal tension cracking, toe crushing with diagonally-oriented cracks, or stair-stepped bed-joint sliding.

- Observed damage is often not linked to points on the force/displacement hysteresis loops.

- Final drift values are not always given; when they are, it is often unclear why the test was stopped and

whether additional stable deformation capacity remained.

- In many tests, the applied axial load varies significantly from the desired nominal value at different times during the test. Thus, lateral capacities can be affected.

4.1.1.1   Rocking

Reference: Anthoine et al. (1995)
Specimen: High wall, first run
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m=6.2 MPa, brick f'_m=16 MPa
  L/h_{eff}=1m/2m= 0.5
  Nominal f_a=0.60 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
  v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
  V_r=68 V_{tc}=65
  V_{bjs1}=73 V_{bjs2}=43
  V_{dt1}=85 V_{dt2}=130
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Rocking at 68 kN
  with drift "d"=0.8%
Actual Behavior: Rocking at 72 kN with test
  stopped at 0.6%. Slight cracks at mid-pier. Axial
  load increased for second run (see below).

- There is no direct test for f'_{dt1}. FEMA 273 equations use v_{me} for f'_{dt}. This gives the value for V_{dt1}. As an additional check, 1/30th of the value of flat-wise compressive strength of the masonry units was also used; this results in the value for V_{dt2}.

Hysteretic response of the high wall, first run.

Reference: Anthoine et al. (1995)
Specimen: High wall, second run
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m=6.2 MPa, brick f'_m=16 MPa
   L/h_{eff}=1m/2m= 0.5
   Nominal f_a= 0.80 MPa
   Fixed-fixed end conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
   v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
   V_r=90 V_{tc}=82
   V_{bjs1}=85 V_{bjs2}=58
   V_{dt1}=104 V_{dt2}=141
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Rocking, then stair-stepped bed-
   joint sliding at a drift of 0.75%
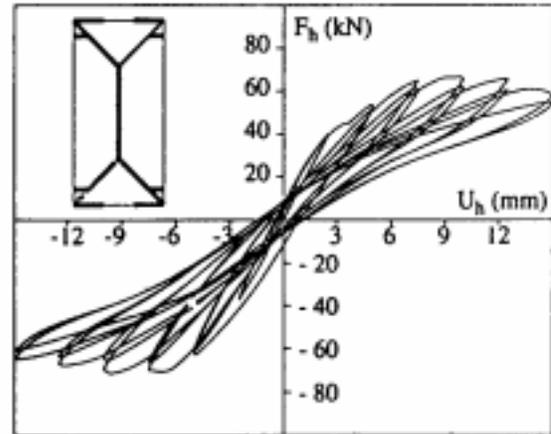
Hysteretic response of the high wall, second run.

Reference: Magenes & Calvi (1995)
Specimen: 3, runs 7-12
Material: Brick
Loading: Shaketable
Provided Information:
   Prism f'_m=8.6 MPa, brick f'_m=18.2 MPa
   L/h_{eff}=1m/2m = 0.5
   Nominal f_a= 0.63 MPa
   Fixed-fixed end conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(1.15+0.57f_a) MPa
   v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
   V_r=71 V_{tc}=70
   V_{bjs1}=189 V_{bjs2}=45
   V_{dt1}=171 V_{dt2}=145
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Rocking at 71 kN with
drift "d" = 0.8%.
Actual Behavior: Rocking at 87 kN with drift of
   1.3% in run 10.
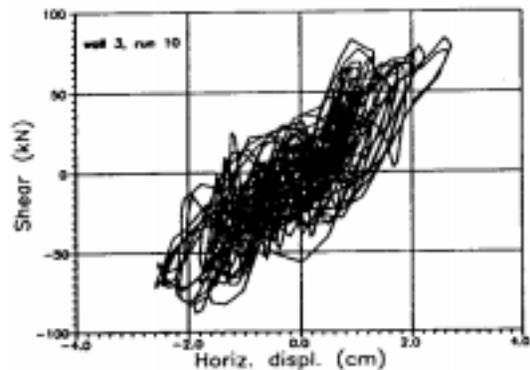
*Shear-displacement curve characterized
by rocking (wall 3, run 10). The figure
does not show final runs 11 and 12.*

Reference: Costley & Abrams (1996)
Specimen: S1 Door Wall
Material: Brick
Loading: 3/8th-scale building on shaketable
Provided Information:
  Prism f'_m=1960 psi, brick f'_m=6730 psi
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*361+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Outer Piers:
  L/h_{eff}=1.44ft/2.67ft =0.54
  Nominal f_a= 33 psi
  Calculated Values (kips):
     V_r=1.0 V_{tc}=1.1
     V_{bjs1}=9.7 V_{bjs2}=1.1
     V_{dt1}=7.2 V_{dt2}=10.3
Inner Pier:
  L/h_{eff}=0.79ft/1.50ft =0.53
  Nominal f_a= 40 psi
  Calculated Values (kips):
     V_r=2.7 V_{tc}=2.9
     V_{bjs1}=15.3 V_{bjs2}=1.8
     V_{dt1}=14.3 V_{dt2}=20.4
FEMA 273 Predicted Behavior of Wall Line: Rocking at 4.7 kips with inner-pier drift "d"=0.5%
ATC-43 Predicted Behavior of Wall Line: Same as FEMA 273
Actual Behavior of the Wall Line:
  Run 14: Rocking up to 8 kips, then stable at 4-6 kips. Drift up to 1.1%.
  Run 15: Rocking at 4-6 kips with drift up to 1.3%
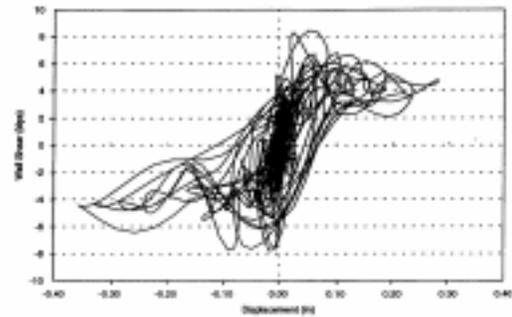
Door-wall shear vs. first-level door-wall displacement from Test Run 14
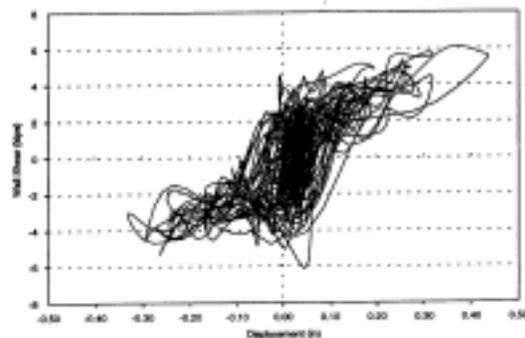
Door-wall shear vs. first-level door-wall displacement from Test Run 15

Reference: Costley & Abrams (1996)
Specimen: S2 Door Wall
Material: Brick
Loading: 3/8th-scale building on shaketable
Provided Information:
Prism f'_m=1960 psi, brick f'_m=6730 psi
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*361+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Outer Piers:
L/h_{eff}=0.79ft/2.67ft =0.30
Nominal f_a= 40 psi
Calculated Values (kips):

V_r=0.4	V_{tc}=0.4
V_{bjs1}=5.5	V_{bjs2}=0.7
V_{dt1}=4.1	V_{dt2}=5.7

Inner Piers:
L/h_{eff}=1.12ft/2.67ft =0.42
Nominal f_a= 48 psi
Calculated Values (kips):

V_r=0.9	V_{tc}=1.0
V_{bjs1}=7.9	V_{bjs2}=1.2
V_{dt1}=6.1	V_{dt2}=8.2

FEMA 273 Predicted Behavior of Wall Line: Rocking at 2.6 kips with inner-pier drift "d"=1.0%
ATC-43 Predicted Behavior of Wall Line: Same as FEMA 273
Actual Behavior of the Wall Line:
Run 22: Rocking at 4 kips, up to a 0.3% drift
Run 23: Rocking at 4 kips, up to a 0.8% drift
Run 24: Rocking at 4 kips, up to a 1.1% drift
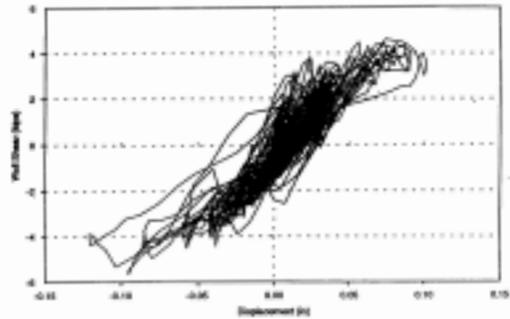
Door-wall shear vs. first-level door-wall displacement from Test Run 22
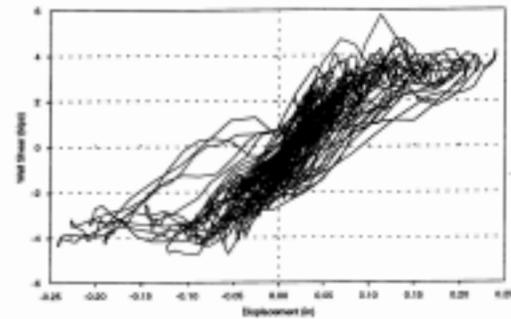
Door-wall shear vs. first-level door-wall displacement from Test Run 23
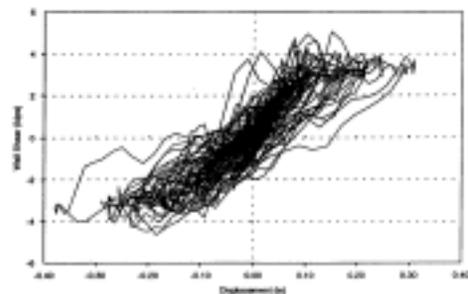
Door-wall shear vs. first-level door-wall displacement from Test Run 24

4.1.1.2   Bed-joint Sliding

Reference: Magenes & Calvi (1992)
Specimen: MI4
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
  L/h_{eff}=1.5m/3m = 0.5
  Nominal f_a= 0.69 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
  v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
  V_r=177 V_{tc}=172
  V_{bjs1}=219 V_{bjs2}=160
  $V_{dt1=}$245 V_{dt2}=360
FEMA 273 Predicted Behavior: Toe crushing at 172 kN
ATC-43 Predicted Behavior: Rocking at 177kN with drift "d" = 0.8%
Actual Behavior: Stair-stepped bed-joint sliding at 153 kN with a final drift of 0.6%

Specimen MI4

Reference: Abrams & Shah (1992)
Specimen: W1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m=911 psi, brick f'_m=3480 psi
  L/h_{eff}=12ft/6ft = 2
  Nominal f_a= 75 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*100+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
  V_r=76 V_{tc}=74
  V_{bjs1}=84 V_{bjs2}=42
  V_{dt1}=149 V_{dt2}=167
FEMA 273 Predicted Behavior: Toe crushing at 74 kips
ATC-43 Predicted Behavior: Flexural cracking/toe crushing/bed-joint sliding with a peak load of 74 kips with "d" drift of 0.4%
Actual Behavior: Bed-joint sliding at 92 kips with test stopped at a drift of 2.4%.
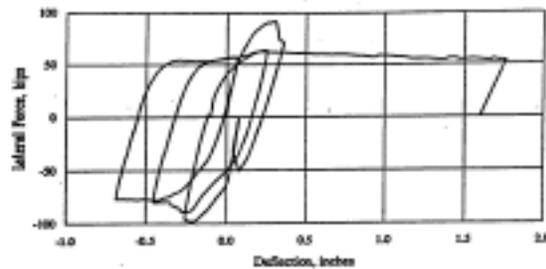
Test Wall W1

Reference: Magenes & Calvi (1995)
Specimen: 5
Material: Brick
Loading: Shaketable
Provided Information:
  Prism f'_m=6.2 MPa, brick f'_m=16 MPa
  L/h_{eff}=1m/1.35m = 0.74
  Nominal f_a= 0.63 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
  v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
  V_r=105 V_{tc}=102
  V_{bjs1}=74 V_{bjs2}=45
  V_{dt1}=97 V_{dt2}=160
FEMA 273 Predicted Behavior: Bed-joint sliding at
  74 kN with "d" drift of 0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Flexural cracking then horizontal
  and stepped bed-joint sliding with peak load of 114
  kN
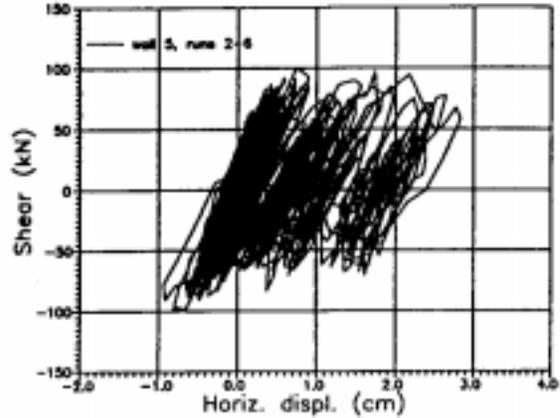
*Shear-displacement curve characterized
by rocking and sliding (wall 5, runs 2-6).
The figure does not show final run 7.*

Reference: Magenes & Calvi (1992)
Specimen: MI2
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
  L/h_{eff}=1.5m/2m = 0.74
  Nominal f_a= 0.67 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
  v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
  V_r=257 V_{tc}=251
  V_{bjs1}=213 V_{bjs2}=155
  V_{dt1}=267 V_{dt2}=399
FEMA 273 Predicted Behavior: Bed-joint sliding at
  213 kN with "d" drift of 0.4%.
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Horizontal bed-joint sliding at top
  course, then stair-stepped bed-joint sliding with a
  peak load of 227 kN and drift of 0.7%
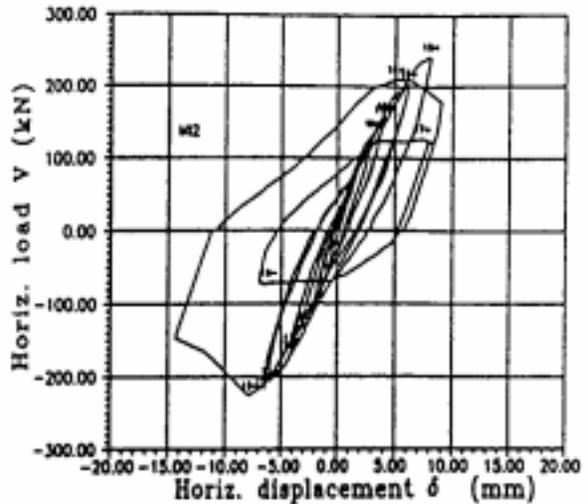
Specimen MI2

4.1.1.3 Rocking/Toe Crushing

Reference: Abrams & Shah (1992)
Specimen: W3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m= 911 psi, brick f'_m= 3480 psi
   L/h_{eff}= 6ft/6ft =1.0
   Nominal f_a= 50 psi
   Cantilever conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.75*100+f_a) psi
   v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
   V_r= 12.6 V_{tc}=12.9
   V_{bjs1}=35 V_{bjs2}=14
   V_{dt1}=69 V_{dt2}=78
FEMA 273 Predicted Behavior: Rocking at 12.6
   kips with drift "d"=0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Rocking at 20 kips then toe crush-
   ing at drift of 0.8%
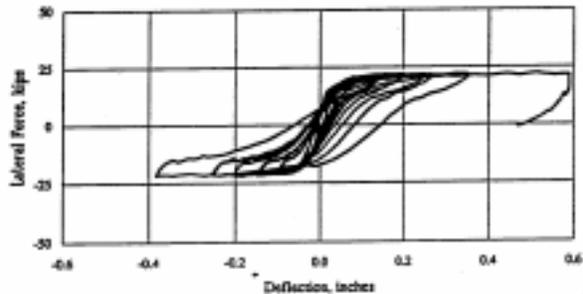
***Test Wall W3: Measured relation between
lateral force and deflection.***

4.1.1.4 Flexural Cracking/Toe Crushing/Bed-Joint Sliding

Reference: Manzouri et al. (1995)
Specimen: W1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m= 2000 psi, brick f'_m= 3140 psi
   L/h_{eff}= 8.5ft/5ft =1.7
   Nominal f_a= 150 psi
   Cantilever conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
   v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
   V_r= 152 V_{tc}=151
   V_{bjs1}=156 V_{bjs2}=99
   V_{dt1}=235 V_{dt2}=172
FEMA 273 Predicted Behavior: Toe crushing at 151
   kips.
ATC-43 Predicted Behavior: Flexural cracking/toe
   crushing/bed-joint sliding with a 151 kip peak load,
   99 kip load for "c" and a "d"drift of 0.4%.
Actual Behavior: Flexural cracking at 88 kips, toe
   crushing then bed-joint sliding at 156 kips, with a
   final drift of 1.3%
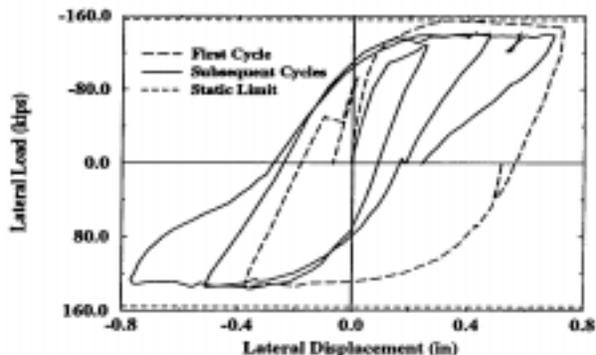
Specimen W1

Reference: Manzouri et al. (1995)
Specimen: W2
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m= 2200 psi, brick f'_m= 3140 psi
  L/h_{eff}= 8.5ft/5ft =1.7
  Nominal f_a= 55 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
  V_r= 56 V_{tc}=60
  V_{bjs1}=93 V_{bjs2}=36
  V_{dt1}=124 V_{dt2}=171
FEMA 273 Predicted Behavior: Rocking at 56 kips.
ATC-43 Predicted Behavior: Flexural cracking/toe
  crushing at 60 kips.
Actual Behavior: Flexural cracking at 31 kips, toe
  crushing at 68 kips, diagonal cracking at 62 kips,
  then bed-joint sliding at 52 kips and below, with a
  final drift of 1.2%
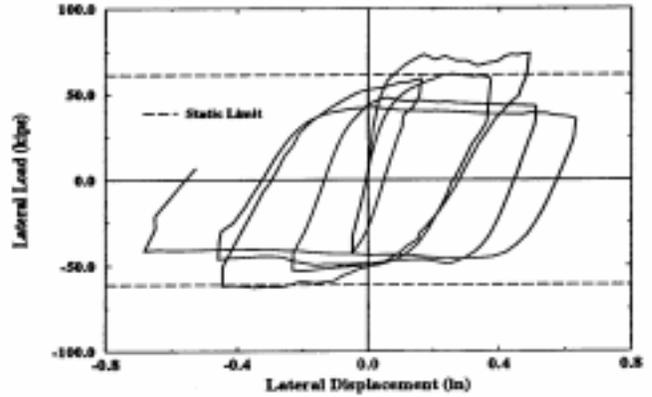
Specimen W2

Reference: Manzouri et al. (1995)
Specimen: W3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m= 2600 psi, brick f'_m= 3140 psi
  L/h_{eff}= 8.5ft/5ft =1.7
  Nominal f_a= 85 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
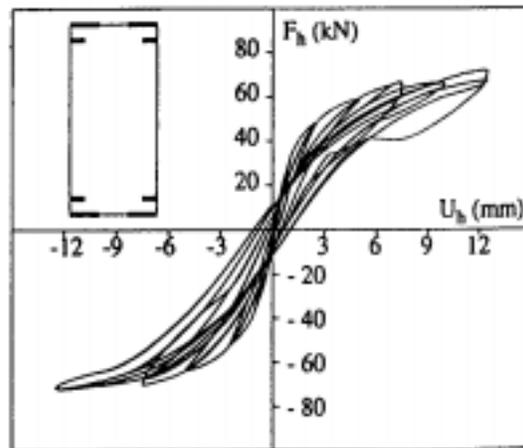**Calculated Values (kips):**
  V_r= 86 V_{tc}=91
  V_{bjs1}=113 V_{bjs2}=56
  V_{dt1}=159 V_{dt2}=187
FEMA 273 Predicted Behavior: Rocking at 86 kips.
ATC-43 Predicted Behavior: Flexural cracking/toe
  crushing/bed-joint sliding with a 91 kip peak load,
  56 kip load for "c" and a "d"drift of 0.4%.
Actual Behavior: Flexural cracking at 55 kips, toe
  crushing at 80 kips, then bed-joint sliding at 80 kips,
  reducing to 56-62 kips, with some final toe crushing
  up to final drift of 0.8%
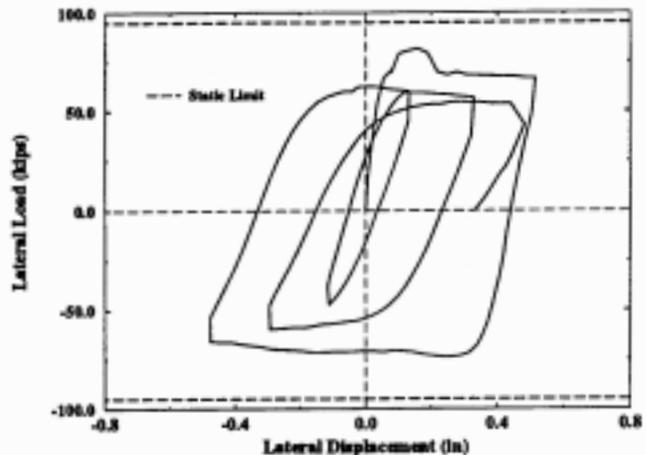
Specimen W3

4.1.1.5 Flexural Cracking/Diagonal Tension

Reference: Anthoine et al. (1995)
Specimen: Low Wall
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m=6.2 MPa, brick f'_m=16 MPa
   L/h_{eff}=1m/1.35m= 0.74
   Nominal f_a= 0.60 MPa
   Fixed-fixed end conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
   v_{me2}=(0.75/1.5)*(0.57f_a) MPa
**Calculated Values (kN):**
   V_r=100 V_{tc}=96
   V_{bjs1}=73 V_{bjs2}=43
   V_{dt1}=94 V_{dt2}=144
FEMA 273 Predicted Behavior: Bed-joint sliding at
   73 kips with "d" drift of 0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Flexural cracking then diagonal
   tension cracking with a peak load of 84 kN and a
   final drift of 0.5%
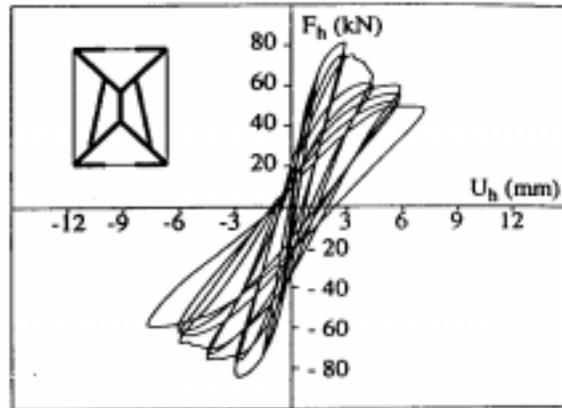
Hysteretic response of the low wall.

Reference: Magenes & Calvi (1992)
Specimen: MI3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
   L/h_{eff}=1.5m/3m = 0.5
   Nominal f_a= 1.245 MPa
   Fixed-fixed end conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
   v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
   V_r=319 V_{tc}=275
   V_{bjs1}=347 V_{bjs2}=288
   V_{dt1}=406 V_{dt2}=427
FEMA 273 Predicted Behavior: Toe crushing
ATC-43 Predicted Behavior: Flexural cracking/
   diagonal tension at 275 kN
Actual Behavior: Flexural cracking then diagonal
   tension cracking with a peak load of 185 kN and a
   final drift of 0.5%
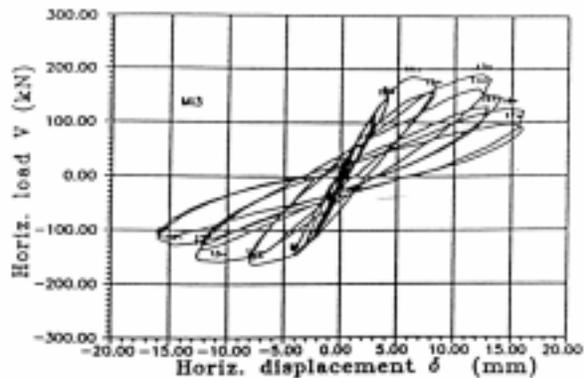
Specimen MI3

Reference: Magenes & Calvi (1995)
Specimen: 8
Material: Brick
Provided Information:
  Prism f'_m=6.2 MPa, brick f'_m=16 MPa
  L/h_{eff}=1m/2m = 0.5
  Nominal f_a= 1.11 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
  v_{me2}=(0.75/1.5)*(0.57f_a) MPa
**Calculated Values (kN):**
  V_r=125 V_{tc}=109
  V_{bjs1}=108 V_{bjs2}=79
  V_{dt1}=137 V_{dt2}=171
FEMA 273 Predicted Behavior: Bed-joint sliding or
  toe crushing.
ATC-43 Predicted Behavior: Bed-joint sliding or
  flexural cracking/diagonal tension at 108-109 kN
Actual Behavior: Flexural cracking then diagonal
  tension cracking with a peak load of 129 kN and a
  final drift of 0.8-1.3%
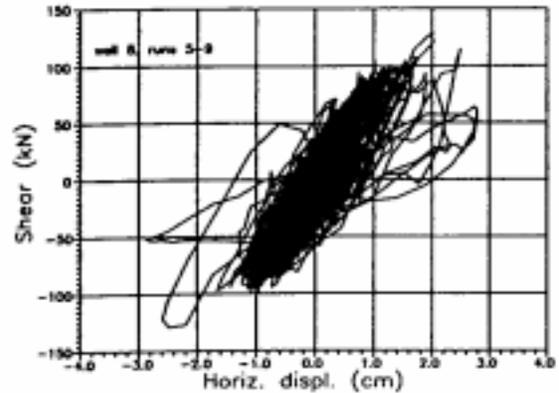
**Brittle collapse due to diagonal cracking
(wall 8, runs 5-9)**

Reference: Magenes & Calvi (1992)
Specimen: MI1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
  Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
  L/h_{eff}=1.5m/2m= 0.75
  Nominal f_a= 1.123 MPa
  Fixed-fixed end conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
  v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
  V_r=432 V_{tc}=383
  V_{bjs1}=319 V_{bjs2}=260
  V_{dt1}=415 V_{dt2}=462
FEMA 273 Predicted Behavior: Bed-joint sliding at
  319 kN with drift "d"=0.4%
ATC-43 Predicted Behavior of Wall Line: Same as
  FEMA 273
Actual Behavior: Flexural cracking then diagonal
  tension at 259 kN, with maximum drift of 0.6%
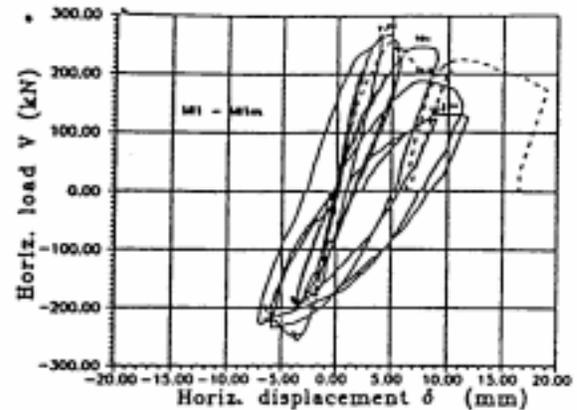
**Test on wall MI1 and MI1m
(dashed line); h = 2m.**

4.1.1.6   Flexural Cracking/Toe Crushing

Reference: Abrams & Shah (1992)
Specimen: W2
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
   Prism f'_m= 911 psi, brick f'_m=3480 psi
   L/h_{eff}= 9ft /6ft = 1.5
   Nominal f_a= 50 psi
   Cantilever conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.75*100+f_a) psi
   v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
   V_r=28 V_{tc}=29
   V_{bjs1}=53 V_{bjs2}=21
   V_{dt1}= 155 V_{dt2}= 175
FEMA 273 Predicted Behavior: Rocking at 28 kips
   with drift "d"=0.3%
ATC-43 Predicted Behavior: Flexural cracking/toe
   crushing at 29 kips
Actual Behavior: Flexural cracking/toe crushing,
   with a maximum capacity of 43-45 kips.
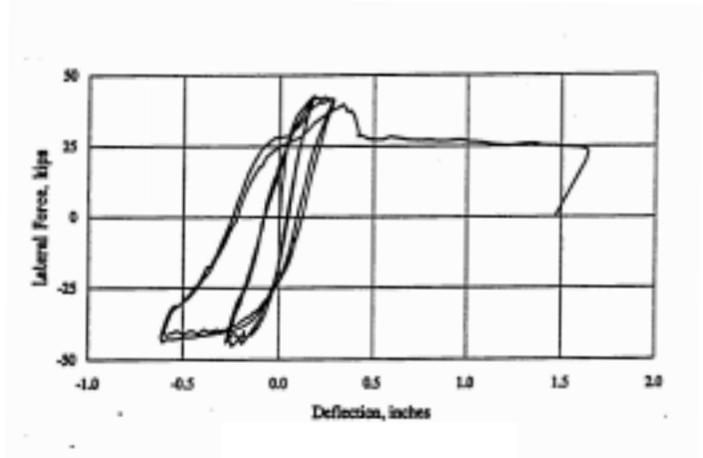
Test Wall W2

Reference: Epperson and Abrams (1989)
Specimen: E1
Material: Brick
Loading: Monotonic
Provided Information:
   Prism f'_m= 1740 psi, brick f'_m=8280 psi
   L/h_{eff}= 7.83ft /6ft = 1.31
   Nominal f_a= 126 psi
   Cantilever conditions
Assumed Values:
   v_{me1}=(0.75/1.5)*(0.75*186+f_a) psi
   v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
   V_r=118 V_{tc}=118
   V_{bjs1}=250 V_{bjs2}=101
   V_{dt1}= 336 V_{dt2}= 533
FEMA 273 Predicted Behavior: Rocking or toe
   crushing
ATC-43 Predicted Behavior: Flexural cracking/toe
   crushing at 118 kips
Actual Behavior: Flexural cracking/toe crushing,
   with a maximum capacity of 120 kips and final drift
   of 0.3%
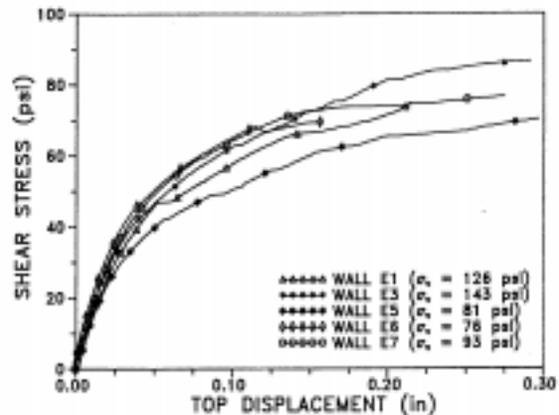
*Summary of measured top level
displacement of the Test Walls
vs shear stress*

Reference: Epperson and Abrams (1989)
Specimen: E3
Material: Brick
Loading: Monotonic
Provided Information:
  Prism f'_m= 1740 psi, brick f'_m=8280 psi
  L/h_{eff}= 9.5ft /6ft = 1.58
  Nominal f_a= 141 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*186+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
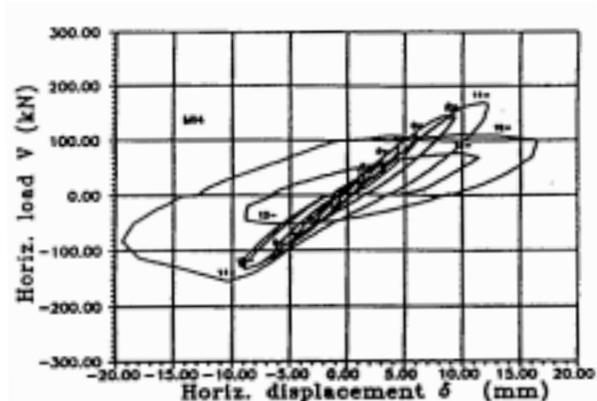 V_r=190 V_{tc}=186
  V_{bjs1}=307 V_{bjs2}=133
  V_{dt1}= 420 V_{dt2}= 635
FEMA 273 Predicted Behavior: Toe crushing at 186 kips
ATC-43 Predicted Behavior: Flexural cracking/toe crushing at 186 kips
Actual Behavior: Flexural cracking/toe crushing, with a maximum capacity of 164 kips and final drift of 0.4%

Reference: Epperson and Abrams (1989)
Specimen: E5
Material: Brick
Loading: Monotonic
Provided Information:
  Prism f'_m= 1740 psi, brick f'_m=8280 psi
  L/h_{eff}= 11.42ft /6ft = 1.90
  Nominal f_a= 81 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*186+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
**Calculated Values (kips):**
  V_r=150 V_{tc}=156
  V_{bjs1}=289 V_{bjs2}=88
  V_{dt1}= 367 V_{dt2}= 680
FEMA 273 Predicted Behavior: Rocking at 150 kips with "d" drift of 0.2%
ATC-43 Predicted Behavior: Flexural cracking/toe crushing at 156 kips
Actual Behavior: Flexural cracking/toe crushing, with a maximum capacity of 154 kips and final drift of 0.4%

Reference: Epperson and Abrams (1989)
Specimen: E6
Material: Brick
Loading: Monotonic
Provided Information:
  Prism f'_m= 1740 psi, brick f'_m=8280 psi
  L/h_{eff}= 11.42ft /6ft = 1.90
  Nominal f_a= 76 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*186+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
  V_r=141 V_{tc}=147
  V_{bjs1}=284 V_{bjs2}=82
  V_{dt1}= 357 V_{dt2}= 675
FEMA 273 Predicted Behavior: Rocking at 141 kips with "d" drift of 0.2%
ATC-43 Predicted Behavior: Flexural cracking/toe crushing at 147 kips
Actual Behavior: Flexural cracking/toe crushing, with a maximum capacity of 150 kips and final drift of 0.2%

Reference: Epperson and Abrams (1989)
Specimen: E7
Material: Brick
Loading: Monotonic
Provided Information:
  Prism f'_m= 1740 psi, brick f'_m=8280 psi
  L/h_{eff}= 11.42ft /6ft = 1.90
  Nominal f_a= 93 psi
  Cantilever conditions
Assumed Values:
  v_{me1}=(0.75/1.5)*(0.75*186+f_a) psi
  v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
  V_r=173 V_{tc}=177
  V_{bjs1}=302 V_{bjs2}=101
  V_{dt1}= 390 V_{dt2}= 692
FEMA 273 Predicted Behavior: Rocking at 173 kips with "d" drift of 0.2%
ATC-43 Predicted Behavior: Flexural cracking/toe crushing at 177 kips
Actual Behavior: Flexural cracking/toe crushing, with a maximum capacity of 157 kips and final drift of 0.4%

4.1.2 Comments on FEMA 273 Component Force/Displacement Relationships

4.1.2.1 Conclusions from Review of the Research and Their Impact on the Evaluation Methodology

As the previous sections indicate, the FEMA 273 methodology leads to successful predictions in certain cases. In other cases, the predictions did not match the observed behavior. To help address this issue, some modifications were made in the Section 7.3 methodology in FEMA 306. Some of these issues and their resolution include:

- Rocking and toe crushing equations often yield very similar values; when they do differ, the lower value does not necessarily predict the governing mode. Section 7.3 in FEMA 306 thus identifies which mode will occur on the basis of aspect ratio, unless the axial stress is very high, since there have been no reported instances of rocking in stocky piers. The $L/h_{eff} > 1.25$ is a somewhat arbitrary threshold based simply on a review of test results.

- Stable rocking generally exceeds the proposed "d" drift value of $0.4h_{eff}/L$. Thus, this value is conservative (see Costley and Abrams, 1996 and Anthoine et al., 1995).

- Rocking does not appear to exhibit the FEMA 273 drop to the "c" capacity value in the above two tests nor, apparently, in the Magenes and Calvi (1995) tests. The only exception is Specimen W3 of Abrams and Shah (1992), which, after rocking for ten cycles at drifts of up to 0.5% ($0.5h_{eff}/L$), was then pushed to 0.8% drift ($0.8h_{eff}/L$) where it experienced toe crushing. The test was stopped at that point. Given the limited number of specimens, it is difficult to determine if this represents the drop from initial load to the "c" level, or a special, sequential mode. For simplicity, this case was combined with the rocking cases, and the "d" drift level was set to account for this level of toe crushing. In most cases, though, rocking capacities will not drop off significantly. The "d" drift value of $0.4h_{eff}/L$ was set based on Costley and Abrams (1996), with some conservatism (Abrams, 1997) to account for Specimen W3. The "c" drift value was conservatively set at 0.6, because of the limited test

data (Abrams, 1997), but aside from Specimen W3, higher "c" values are probably likely.

- There are few pure bed-joint sliding tests. Specimen W1 of Abrams and Shah (1992) is one example, and Specimens MI2 and MI4 of Magenes and Calvi (1992) appear to be examples as well. The drop in lateral strength appears to occur at about 0.3-0.4% drift in W1 and MI4, so the proposed "d" value of 0.4 seems reasonable. The "c" of 0.6 also seems reasonable. The capacity for bed-joint sliding is based on the bond-plus-friction strength. After cracking, the bond capacity will be eroded, and the strength is likely to be based simply on the friction portion of the equation. Cyclic in-place push tests show this behavior; so does Specimen W1 of Abrams and Shah (1992). One could argue that the second cycle backbone curve of FEMA 273 (which, by definition, goes into the nonlinear, post-cracking range) should be limited only to the frictional capacity. But in many cases, other modes will be reached before the full bed-joint sliding capacity is reached. In some of these cases, interestingly, bed-joint sliding occurs after another mode has occurred. Manzouri et al. (1995), for example, show sequences such as initial toe crushing that progresses to bed-joint sliding at higher drift values. One explanation is that toe crushing degenerated into bed-joint sliding because the toe crushing and initial bed-joint sliding values were quite close. See Section 4.1.2.2 for further explanation.

- Mixed modes or, more accurately, sequences of different behavior modes are common in the experiments.

4.1.2.2 The Bed-Joint Sliding and Flexural Cracking/Toe Crushing/Bed-Joint Sliding Modes

The model of bed-joint sliding used in this document is shown in Figure 4-1. For estimating the strength and deformation capacity of the undamaged bed-joint sliding mode, FEMA 273 was used. The idealized relationship has a plateau at the bed-joint capacity V_{bjs1}, which includes the bond and friction components. After bond is lost, the residual strength is limited to 60% of V_{bjs1}. The actual backbone curve is likely to be smoother than the idealized model, since the loss of bond does not occur all at once in the entire masonry section. Instead, more heavily stressed portions crack, and shear demand is redistributed to the remaining
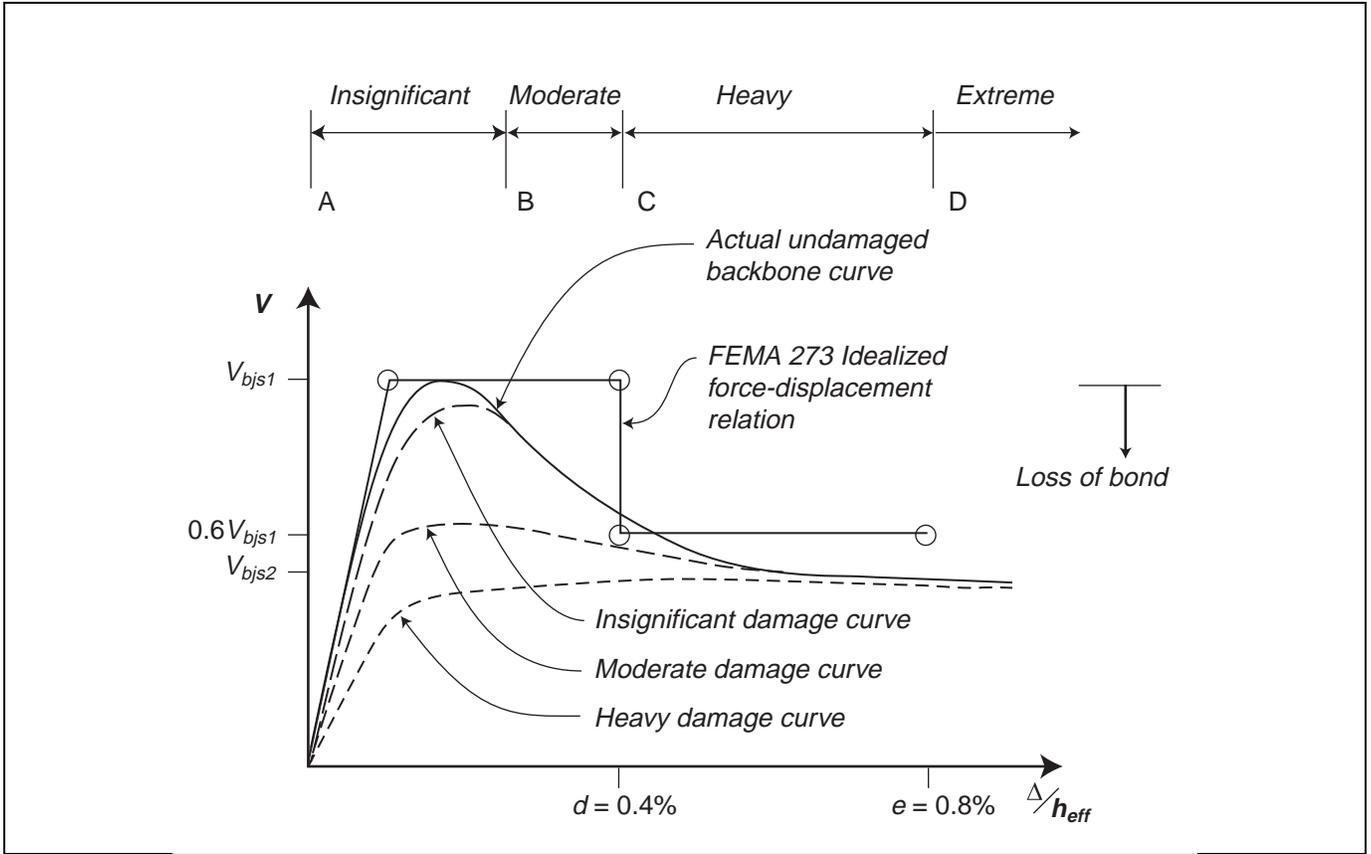
Figure 4-1   Bed-joint sliding force/displacement relationship

sections. The actual residual strength could be higher or lower than $0.6V_{bjs1}$. One measure of the residual capacity is V_{bjs2}.

Figure 4-1 also shows the assumed changes to the force/displacement relationship following the damaging event. *Insignificant* damage is characterized by displacement during the damaging event that is between points A and B. Loss of bond is limited. Following the damaging event, the dashed "Insignificant Damage Curve" represents the force/displacement relationship. For damaging events that reach levels of initial displacement beyond point B, greater loss of bond occurs, and the subsequent damage curve achieves a lower strength. Eventually, with initial displacements beyond point C, the entire bond is lost and only friction remains. Thus, future cycles will no longer be able to achieve the original V_{bjs1} level, reaching only the V_{bjs2} level. With significant cyclic displacements, some erosion of the crack plane and deterioration of the wall

is likely to lead to a small reduction in capacity below the V_{bjs2} level.

The varying level of bed-joint sliding strength is assumed in this document to be a possible explanation for some of the observed testing results in stocky walls, in particular results such as (1) Specimen W1 of Abrams and Shah (1992), in which bed-joint sliding was the only mode observed; (2) Manzouri et al. (1995), in which toe crushing behavior was followed by bed-joint sliding; and (3) Epperson and Abrams (1989), in which toe crushing was not followed by sliding. Figure 4-2 helps to explain the hypothesis.

In the top set of curves, toe-crushing strength substantially exceeds the V_{bjs1} level. As displacement occurs, the bed-joint sliding capacity is reached first, and it becomes the limit state. If displacement is such that *heavy* damage occurs, then in subsequent cycles, the strength will be limited to the V_{bjs2} level.

1.25 V_{bjs1} < V_{tc} :
Bed-joint sliding

V_{tc} — Toe crushing
V_{bjs1} — Bed-joint sliding
V_{bjs2}

Initial Force/Displacement Relationship

Bed-joint sliding

Heavy Damage

V_{bjs2} < 0.75 V_{bjs1} < V_{tc} < V_{bjs1} :
Flexural yield/Toe crushing/Bed-joint sliding

V_{bjs1} — Toe crushing
V_{tc} — Bed-joint sliding
V_{bjs2} — Composite curve

Initial Force/Displacement Relationship

Bed-joint sliding

Heavy Damage

0.75 V_{bjs1} < V_{tc} :
Toe crushing

V_{bjs1} — Toe crushing
— Bed-joint sliding
V_{bjs2} — Composite curve

Initial Force/Displacement Relationship

Toe crushing

Heavy Damage

Figure 4-2   ***Relationship Between Toe Crushing and Bed-Joint Sliding***

In the second set of curves, toe-crushing and initial bed-joint sliding strengths are similar. As displacement occurs, the toe-crushing strength is reached first, cracking and movement occur within the wall, some of the bond is lost, and the wall begins to slide. The initial force/displacement curve is thus similar to that for bed-joint sliding, except that the peak is limited by the toe-crushing strength. If displacement is such that *Heavy* damage occurs, then in subsequent cycles, the strength

will be limited to the V_{bjs2} level. This is one possible explanation for the Manzouri et al. (1995) tests.

In the third set of curves, toe-crushing strength is substantially lower than initial bed-joint sliding strength and the ductile mechanism of sliding is not achieved. This is one possible explanation for the Epperson and Abrams (1989) results, in which mortar shear strength was much higher and ductility was lower.

Section 7.3.2 in FEMA 306 makes use of the above hypotheses; cutoff values for the middle set of curves were based in part on review of the results shown in Section 4.1.1. Results are promising, but additional testing and verification of other tests should be done.

4.1.2.3 Out-of-Plane Flexural Response

The most comprehensive set of testing done to date on the out-of-plane response of URM walls was part of the ABK program in the 1980s, and it is documented in ABK (1981c). Input motions used in the ABK (1981c) were based on the following earthquake records: Taft 1954 N21E, Castaic 1971 N69E, Olympia 1949 S04E, and El Centro 1940 S00E. They were scaled in amplitude and were processed to represent the changes caused by diaphragms of varying stiffness to produce the final series of 22 input motion sets. Each set has a motion for the top of the wall and the bottom of the wall. Peak velocities range up to 39.8 in/sec; accelerations, up to 1.42g; and displacements, up to 9.72 inches. In ABK (1984), the mean ground input velocity for UBC Seismic Zone 4 was assumed to be 12 in/sec. For buildings with crosswalls, diaphragm amplification would increase this about 1.75-fold, to 21 in/sec. For buildings without crosswalls, wood roofs were assumed to have a velocity of about 24 in/sec and floors about 27 in/sec.

Since 1981, a significant number of ground motion records have been obtained, including a number of near-field records. In several instances, recent recordings substantially exceed the 12 in/sec value and even exceed the maximum values used by ABK (1981c). Of particular concern are near-field pulse effects and whether they were adequately captured by the original testing. When site-specific spectra and time histories that incorporate these effects are available, it may be possible to address this issue using the original research.

4.1.3 Development of λ-factors

One of the central goals of this document is to develop a method for quantitatively characterizing the effect of damage on the force/displacement relationship of wall components. Ideally, the most accurate approach would be to have two sets of cyclic tests for a component. One test would be of an initially undamaged wall displaced to failure. The second set would include walls initially displaced to various levels of damage (to represent the "damaging event") and then retested to failure. This would allow for direct determination of the λ-factors

contained in the Component Guides in FEMA 306. Unfortunately, as noted in Section 4.1.1 there have been almost no experimental tests done on damaged URM walls; typically, tests were done on undamaged walls and either stopped or continued only after the damaged wall was repaired.

In the absence of test results on damaged walls, hysteresis curves of initially undamaged walls were reviewed. In reviewing these tests, the goal was to characterize how force/displacement relationships changed from cycle to cycle as displacement was increased. Early cycles were considered to represent "damaging" events, and subsequent cycles represented the behavior of an initially-damaged component. Particular attention was given to tests in which multiple runs on a specimen were performed. In these cases, initial runs (representing not just a damaging cycle, but a damaging earthquake record) were compared with subsequent runs to determine the extent of strength and stiffness deterioration.

Using these tests, the following general approaches were used to estimate λ-factors for this project. The reloading stiffnesses (i.e., the stiffness observed moving from the fourth quadrant to the first) at different cycles or different runs were compared to the initial stiffness to determine λ_K. This variable is estimated to be the ratio of stiffness at higher cycles to the initial stiffness. The assumption made is that if testing had been stopped and the displacement reset to zero and then restarted, the stiffness of the damaged component would have been similar to the reloading stiffness. See Figure 4-3 for an example.

For determining λ_Q, the approach shown in Figure 4-1 and discussed in the previous section is applied where appropriate to determine λ_Q, the ratio of strength at higher cycles to initial strength. The loss of strength is roughly equal to the capacity at high drift levels divided by the peak capacity. FEMA 273 describes both deformation-controlled and force-controlled modes. In a purely force-controlled mode, there is, by definition, little or no ductility. Deformation progresses until a brittle failure results. Thus, there are few, if any, damage states between *Insignificant* and *Extreme*, and there would be little, if any, post-cracking strength. Further, until a brittle mode occurs, the component would be expected to be minimally affected by previous displacement. Review of available hysteresis curves shows, though, that even modes defined as force-
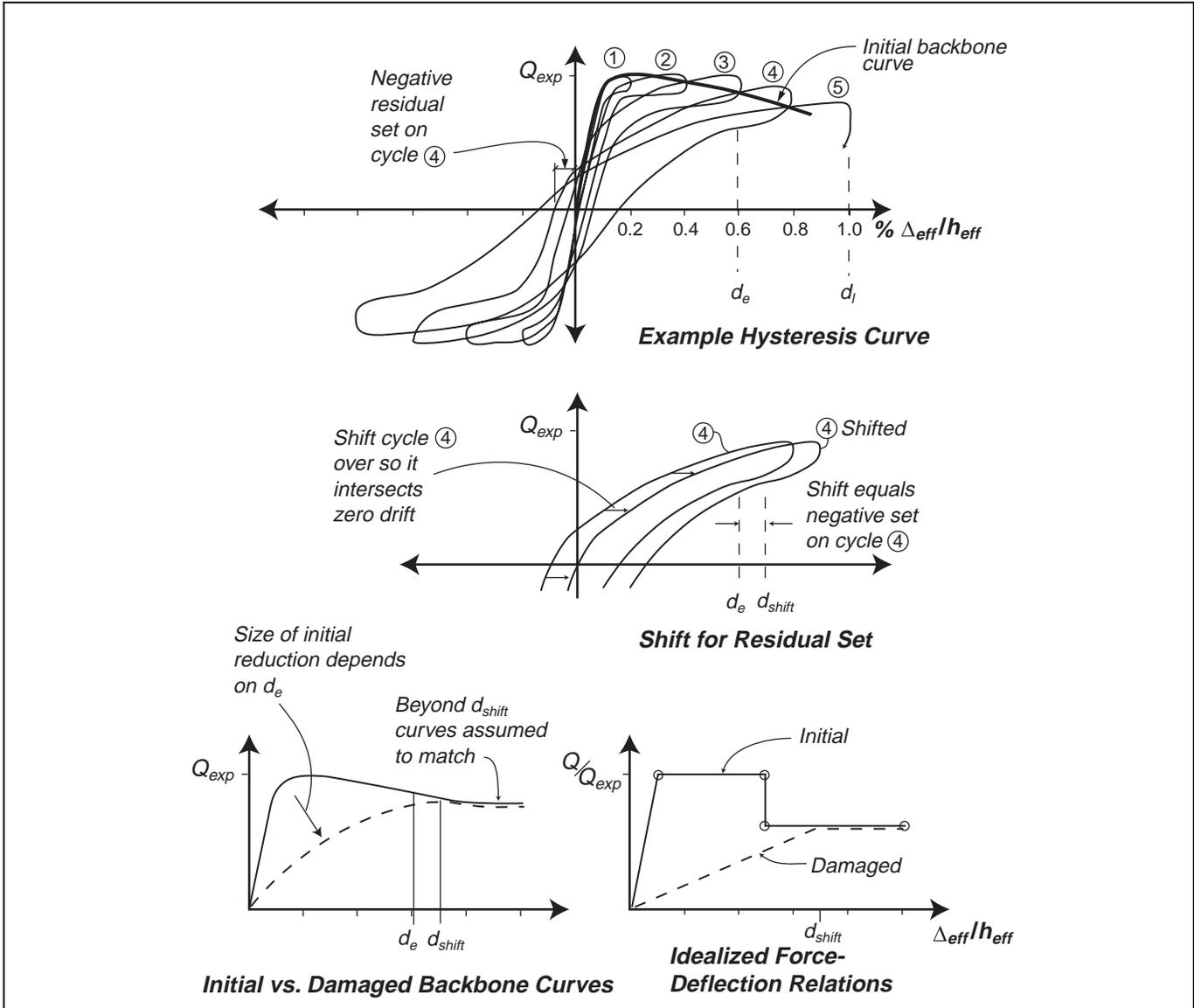
Figure 4-3   Developing the initial portion of the damaged force/displacement relationship

controlled by FEMA 273 (such as diagonal tension) do have some residual strength.

There is little available information for determining λ_Δ, because retesting of damaged components to failure has not been done. Values were estimated using engineering judgment. In most cases, less-ductile modes are assumed to have higher λ_Δ values, even at higher damage levels. The basis of this assumption is the idea that in more-ductile modes, λ_Δ is assumed to be somewhat more dependent on cumulative inelastic deformation. In more-ductile modes, the available

hysteretic energy has been dissipated in part by the damaging earthquake, and there is less available in the subsequent event. The result is the final displacement that can be achieved is reduced.

Values for $\lambda_K{}^*$, $\lambda_Q{}^*$, and $\lambda_\Delta{}^*$ are based, where possible, on tests of repaired walls. The values in URM1F, for example, are set at 1.0 because the hysteresis curves of repaired walls were equal to or better than those of the original walls. In most other cases, repairs typically involve injection of cracks, but since microcracking can never be fully injected, it may not be possible to restore

complete initial stiffness. In the bed-joint sliding modes without tests, it was assumed that the strength could not be fully restored by injection, because the horizontal crack planes are closed and bond cannot be restored in these locations. It is important to recognize that injection of walls with many cracks or unfilled collar joints and cavities, may enhance strength, but it may also lead to less ductile behavior, because other modes may then occur prior to bed-joint sliding.

Values for $\lambda_{h/t}$ are based on a review of the ABK (1981c) document, the model proposed in Priestley (1985), and engineering judgment. At low levels of damage, the portions of wall between the crack planes are essentially undamaged, and the effective thickness,

t, remains unchanged. At higher levels of damage, deterioration, crushing, and spalling of the corners of the masonry at crack locations reduces the effective thickness and the ability of the wall to resist movements imparted by the diaphragm.

4.2   Tabular Bibliography for Unreinforced Masonry

Table 4-1 contains a brief description of the key technical reports that address specific reinforced masonry component behavior. The component types and their behavior modes are indicated. The full references can be found in Section 4.4.

Table 4-1   *Summary of Significant Experimental Research or Research Summaries*

Reference	Specimen/Loading	Aspect Ratio (L/h_{eff})	Axial Stress (f_a in psi)	Predictive Equations	Repair	Component-Type	a	b	c	d	e	f	g	h	i	j	k	l	m	n
Abrams (1992)	Based on Abrams and Shah (1992) and Epperson and Abrams (1989)			Strength	None	URM1								•						
Abrams and Shah (1992)	3 cantilever brick piers with reversed static-cyclic loading	2 1.5 1	75 50 50	Strength	None	URM1		•			•			•						
ABK (1981c)	22 specimens with dynamic out-of-plane loading, including brick, grouted and ungrouted clay and concrete block	*h/t* from 14.0-25.2	2-23	None	Ferrocement surface coating on 2 specimens	URM1													•	
Anthoine et al. (1995)	3 brick piers in double curvature with reversed static cyclic loading	0.5 0.5 0.74	87 87 116	None	None	URM2	• •	•					•							
Costley and Abrams (1996b)	2 3/8th-scale brick buildings on shake table, each with two punctured walls lines in the in-plane direction	0.54-0.84 0.53-0.74 0.30-0.40 0.96-1.50	33-36 40-48 40-48 33-36	Strength	None	URM2	• • •						•							
Epperson and Abrams (1989)	5 cantilever brick piers with monotonic loading	1.31 1.58 1.90 1.90 1.90	126 143 81 76 93	Strength	None	URM1								• • • • •						
Kingsley et al. (1996)	1 2-story, full-scale brick building with reversed static-cyclic loading	na	na	None	None	URM2														•
Magenes and Calvi (1992)	4 brick piers in double curvature with reversed static cyclic loading	0.75 0.75 0.5 0.5	163 97 181 100	Strength	None	URM2		• •						• •						

Behavior Modes Addressed[1]

Table 4-1   Summary of Significant Experimental Research or Research Summaries (continued)

Reference	Specimen/Loading	Aspect Ratio (L/h_{eff})	Axial Stress (f_a in psi)	Predictive Equations	Repair	Component-Type	a	b	c	d	e	f	g	h	i	j	k	l	m	n
Magenes and Calvi (1995)	8 brick piers in double curvature tested on a shake table, some run multiple times with varying axial load	0.74	59		None	URM2	•	•												
		0.74	68				•	•												
		0.74	152				•						•							
		0.5	62				•													
		0.5	91				•													
		0.5	149				•													
		0.5	160				•													
		0.74	91									•								
		0.74	161										•							
		0.5	91				•													
		0.5	173																	
		0.5	161										•				•			
Manzourzi et. al. (1995)	4 virgin brick piers with reversed static-cyclic loading, 3 cantileved and 1 pair of piers with spandrels	1.7	150	Sophisticated finite-element modelling	Repair techniques include grout injection, pinning, and addition of rebar-filled chases	URM1						•								
		1.7	55			URM1						•								
		1.7	85			URM1						•								
		1.27	70			URM2														•
Rutherford & Chekene (1997)	Contains extensive set of research summaries of URM enhancement	na	na	Uses FEMA 273 and provides equations for enhanced walls	Grout and epoxy injection, surface coatings, adhered fabrics, shotcrete, reinforced and post-tensioned cores, infilled openings, enlarged openings, and steel bracing															
Tomasevic and Weiss (1996)	4 1/4-scale brick buildings on shake table	na	na	None	Compares effectiveness of various wall-diaphragm ties															•

[1]Behavior Mode:
a Wall-pier rocking
b Bed-joint sliding
c Bed-joint sliding at wall base
d Spandrel joint sliding
e Rocking/toe crushing

f Flexural cracking/toe crushing/bed-joint sliding
g Flexural cracking/diagonal tension
h Flexural cracking/toe crushing
i Spandrel unit cracking
j Corner damage

k Preemptive diagonal tension
l Preemptive toe crushing
m Out-of-plane flexural response
n Other: Includes complex modes and those reported as having "diagonal cracking"

4.3 Symbols for Unreinforced Masonry

Symbols used in the unreinforced masonry sections of FEMA 306 and 307 are the same as those given in Section 7.9 of FEMA 273 except for the following additions and modifications.

C — Resultant compressive force in a spandrel, lb

L_{sp} — Length of spandrel, in.

M_{spcr} — Expected moment capacity of a cracked spandrel, lb-in.

M_{spun} — Expected moment capacity of an uncracked spandrel, lb-in.

V_{spcr} — Expected diagonal tension capacity of a cracked spandrel, lb

V_{spun} — Expected diagonal tension capacity of an uncracked spandrel, lb

NB — Number of brick wythes in a spandrel

NR — Number of rows of bed joints in a spandrel

T — Resultant tensile force in a spandrel, lb

V_{bjs1} — Expected shear strength of wall or pier based on bed joint shear stress, including both the bond and friction components, lb

V_{bjs2} — Expected shear strength of wall or pier based on bed joint shear stress, including only the friction component, lb

V_{sp} — Shear imparted on the spandrel by the pier, lb

V_{dt} — Expected shear strength of wall or pier based on diagonal tension using v_{me} for f'_{dt}, lb

V_{tc} — Expected shear strength of wall or pier based on toe crushing using v_{me} for f'_{dt}, lb

W_w — Expected weight of a wall, lb

b_{effcr} — Effective length of interface for a cracked spandrel, in.

b_{effun} — Effective length of interface for an uncracked spandrel, in.

b_h — Height of masonry unit plus bed joint thickness, in.

b_l — Length of masonry unit, in.

b_w — Width of brick unit, in.

d_{sp} — Depth of spandrel, in.

d_{effcr} — Distance between resultant tensile and compressive forces in a cracked spandrel, in.

d_{effun} — Distance between resultant tensile and compressive forces in an uncracked spandrel, in.

f'_{dt} — Masonry diagonal tension strength, psi

v_{bjcr} — Cracked bed joint shear stress, psi

v_{bjun} — Uncracked bed joint shear stress in a spandrel, psi

v_{ccr} — Cracked collar joint shear stress in a spandrel, psi

v_{cun} — Uncracked collar joint shear stress in a spandrel, psi

β — =0.67 when $L/h_{eff}<0.67$, $=L/h_{eff}$ when $0.67 \leq L/h_{eff} \leq 1.0$, and = 1.0 when $L/h_{eff}>1$

Δ_s — Average slip at cracked spandrel (can be estimated as average opening width of open head joint), in.

ε — Factor for estimating the bond strength of the mortar in spandrels

γ — Factor for coefficient of friction in bed joint sliding equation for spandrels

η — Factor to estimate average stress in uncracked spandrel. Equal to NR/2 or, for more sophistication, use $\Sigma_{i=1,NR} [(d_{sp}/2 - b_h(i))/(d_{sp}/2 - b_h)]$

$\lambda_{h/t}$ — Factor used to estimate the loss of out-of-plane wall capacity to damaged URM walls

μ_Δ — Displacement ductility demand for a component, used in FEMA 306, Section 5.3.4, and discussed in Section 6.4.2.4 of FEMA 273. Equal to the component deformation corresponding to the global target displacement, divided by the effective yield displacement of the component (which is defined in Section 6.4.1.2B of FEMA 273).

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5. Infilled Frames

5.1 Commentary And Discussion

There is a wealth of experimental data reported in the literature on infilled frames. Unfortunately, only a limited amount of the research has been performed under *cyclic* loading and conducted on specimens that reflect U.S. construction practice. For these test results, it is evident that infilled frames can possess stable hysteresis loops and continue to carry substantial lateral loads at significant interstory drifts. This is true in spite of the highly damaged appearance and even complete loss of some of the masonry units within an infill panel.

Most experimental results on infilled-frame systems show a mixture of behavior modes that take place at various stages of loading. At low interstory drift levels (0.2% - 0.4%), corner crushing and some diagonal cracking in the panel tend to occur first. This is followed by frame yielding (0.5% - 1.0% interstory drift) and possible bed-joint sliding. As the drift amplitude increases beyond about 1%, cracking in the infill panel becomes more extensive, along with further frame damage. The frame damage takes the form of cracking, crushing, and spalling of concrete in the case of reinforced concrete frames or prying damage to bolted semi-rigid connections in steel frames. The coexistence of several behavior modes makes it difficult to determine what λ-factors should be used for quantitative strength and deformation analysis. Therefore, it is necessary to resort to individual component tests to assess λ-values. The results of experiments conducted by Aycardi et al. (1994) are illustrative of the performance of nonductile reinforced concrete frames. These tests give results for each of the failure modes (except column shear).

In the experimental studies on infilled frames by Mander et al. (1993a,b), steel frames were used and were instrumented with numerous strain gauges so the behavior of the frame could be uncoupled from the behavior of the infill panel. It was, therefore, possible to plot the net lateral load-drift capacity of the brick masonry infill panel. These results were helpful in identifying the λ-factors for corner crushing, diagonal cracking and general shear-failure behavior modes for masonry. The bed-joint sliding behavior mode tends to occur mostly in steel frames with ungrouted/unreinforced masonry infill with low panel height-to-length aspect ratios. The experimental results of Gergely et al. (1994) were useful for identifying λ-factors for this behavior mode.

When investigating the out-of-plane behavior of infilled frame panels, it is difficult to enforce a complete failure, as evidenced by recent tests by Angel and Abrams (1994). It should be noted that these investigators first loaded their specimens in-plane before conducting their out-of-plane tests. Results of this study indicate that lateral strength capacity is generally well in excess of 200 psf. Thus, it is unlikely that out-of-plane failure should occur for normal infill height-to-thickness aspect ratios. These results suggest that if an out-of-plane failure is observed in the field, then some other (in-plane) behavior mode has contributed to the failure of the infill.

Dealing with infill panels with openings is difficult due to the many potential types of openings that may occur in practice. Evidently, when openings are present, the strength capacity is bounded by that of bare frame (lower bound) and that of a system with solid infill panels (upper bound). Although these results are derived from monotonic tests, they suggest that the deformation capacity is not impaired if openings exist.

5.1.1 Development of λ-Factors for Component Guides

The Component Damage Classification Guides and component modification factors (λ-factors) for infilled frames were based on an extensive review of research in the area of both nonductile reinforced concrete frames, as well as masonry structures. The principal references used in this work are listed in the tabular bibliography presented in Section 5.2. For each component behavior mode, three types of λ-factors are used: stiffness reduction factors (λ_K), strength reduction factor (λ_Q) and a displacement reduction factor (λ_D). Description of how each of these λ-factors were derived from experimental evidence and theoretical considerations is presented in what follows.

5.1.2 Development of Stiffness Deterioration—λ_K

As the displacement ductility of a member progressively increases, the member also softens. Even though the strength may be largely maintained at a nominal yield level, softening is manifest in the form of stiffness reduction. The degree of softening is generally related to the maximum displacement ductility the member has previously achieved.

There are several analytical models that can be used to give guidance on how one can assess the degree of softening in an element. For example, Chang and Mander (1994) describe several computational hysteretic models calibrated for reinforced concrete components. Utilizing their information obtained from a calibrated modified Takeda model, the λ_K-factor for stiffness reduction can be related by the following relationship:

$$\lambda_K = \left(\frac{\Delta_{\max}}{\Delta_y}\right)^{-\alpha} = \left(\mu_\Delta\right)^{-\alpha} \tag{5-1}$$

where $\Delta_{\max}$ = maximum displacement in the displacement history, Δ_y = yield displacement, μ_Δ = displacement ductility factor, and α = an experimentally calibrated factor that is material- or specimen-dependent.

Strictly, α should be established on a component-by-component basis. However, for reinforced concrete components there is a range of values from $\alpha = 0.25$ to $\alpha = 1$ that may be applicable, $\alpha = 0.5$ being typical for most specimens. Well detailed members tend to have low α values, whereas higher α values are common for poorly detailed members. Although specific research on infill panels is not developed to the same extent, it seems reasonable that similar trends would be found for these components.

5.1.3 The Determination of λ_Q for Strength Deterioration

In structural elements not specifically designed for seismic resistance, there is generally a lack of adequate transverse reinforcement necessary to provide adequate confinement and shear resistance. As a result, under reversed cyclic loading the strength of such elements deteriorates progressively. Furthermore, if the non-seismically designed frame elements have inadequate anchorage for the reinforcing steel, there can be a gradual loss in strength and then a sudden drop in strength when the anchorage zone or lap splice zone fails. An energy approach can be used to assess the loss of strength in a reinforced concrete column or beam element where inadequate transverse reinforcement is found. The energy-based approach advanced by Mander and Dutta (1997) has been used in developing this process. A summary of the underlying theoretical concepts is given below.

Assuming the moment capacity contributed by the concrete is gradually consumed by the propagating level of damage, then at the end of the i-th cycle it can be

shown that the reduced strength $F_i = \lambda_Q F_n$ can be evaluated through

$$\lambda_Q = \frac{F_i}{F_n} = 1 - \frac{M_c}{M_n}\Sigma D_{ci} = 1 - \frac{M_c}{M_n}\frac{\Sigma\theta_{pi}}{\Sigma\theta_{PC}} \tag{5-2}$$

in which ΣD_{ci} = accumulated damage, $\Sigma\theta_{ci}$ = cumulative plastic drift, M_n = nominal moment capacity, M_c = the moment generated by the eccentric concrete stress block and $\Sigma\theta_{PC}$ = cumulative plastic rotation capacity considering concrete fatigue alone. Using energy concepts where it is assumed that the finite energy reserve of an unconfined concrete section is gradually consumed to resist the concrete compression force, a work expression can be formulated as

$$EWD = IWD \tag{5-3}$$

where EWD = external work done on the section by the concrete compression force defined by the left hand side of the equation below, and IWD = internal work or energy absorption capacity of the section defined by the right hand side of the following equation

$$C_c \times \left(\phi_p \frac{c}{2}\right) \times 2N_c = A_g \int_0^{\varepsilon_{cu}} f_c d\varepsilon \tag{5-4}$$

in which C_c = concrete compression force, ϕ_p = plastic curvature, c = neutral axis depth, $2N_c$ = total number of reversals and A_g = gross area of the concrete section. The integral in the above expression actually denotes the finite energy capacity of an unconfined concrete section which in lieu of a more precise analysis, can be approximated as $0.008\, f_c'$. Note also that the term in brackets in the above equation denotes the plastic strain at the location of the concrete compression force.

Assuming that in a cantilever column the plastic rotation is entirely confined to the plastic hinge zone (of length L_p), using the moment-area theorem and rearranging terms in the above equation, it is possible to solve for the cumulative plastic drift capacity as

$$\Sigma\theta_{PC} = \frac{0.016\left(\dfrac{L_p}{D}\right)}{\left(\dfrac{C_c}{f_c'A_g}\right)\left(\dfrac{c}{D}\right)} \tag{5-5}$$

where $\Sigma\theta_P = 2N_c\theta_p$ is the *cumulative plastic drift* defined as the sum of all positive and negative drift amplitudes up to a given stage of loading; and D = overall depth/diameter of the column.

The concrete damage model described so far is generally applicable to beam and/or column elements with adequate bonding between the longitudinal reinforcement and the surrounding concrete. Thus following Equation 5-2, the concrete strength continues to decay until the moment capacity of the eccentric concrete block is fully exhausted. At this point the residual moment capacity entirely consists of the steel contribution. This is schematically portrayed in Figure 5-1a. However, more often than not, older buildings possess lap splice zones at their column bases. Such splices are not always equipped with adequate lap length to ensure proper development of bond strength. The lap splice thus becomes the weak point in the column which shows a drastic reduction in the strength almost immediately following the lap splice failure. This is depicted in Figure 5-1b where the bond failure in the lap splice is assumed to occur over one complete cycle. The residual strength immediately after F_i is determined by the extent of confinement around the lap splice, if any. Subsequently the lateral strength is entirely dependent on the performance of pure concrete which continues to decay following the same Equation 5-2 until the residual rocking strength F_r is obtained.

This theory has been validated with experimental results as shown in Figures 5-1c and 5-1d. In Figure 5-1c, the lateral strength envelope is compared with test results with instances of unconfined concrete failure only. In Figure 5-1d, the strength envelope is plotted for column specimen with a clear indication of lap splice failure. Satisfactory agreement between theory and experiment is observed.

Therefore, with the mechanism of failure and the progression of strength deterioration clearly identified and quantified, it is possible to assess, analytically, λ_Q factors for reinforced concrete elements with specific detailing. The research has not been developed to the same extent for infill panels, although an examination of test results indicates that similar trends are present.

5.1.4 Development of λ_D—Reduction in Displacement Capability

The reduction in displacement capability is more difficult to ascertain from traditional, quasi-static, reversed-cyclic-loading, laboratory tests on members. Generally such tests are conducted using two cycles at each ductility factor (or drift angle percentages) of ± 1, ± 2, ± 6... until failure occurs. The reduction in displacement capacity depends on the severity of the

previous loading history—that is, the amount of energy absorbed with respect to the total energy absorption capacity. Strictly this cannot be ascertained without resorting to fatigue type of testing.

Mander et al. (1994, 1995) and Mander and Dutta (1997) have shown that the displacement capability of structural concrete and steel elements follows a well-known Manson-Coffin fatigue relationship that can be written in displacement ductility terms as follows:

$$\mu_\Delta = \mu_m N_f^c \qquad (5\text{-}6)$$

where N_f = number of equi-amplitude cycles required to produce failure at ductility amplitude μ_Δ; μ_m = monotonic ductility capacity; and c = fatigue exponent. Typical values of the latter are $c = -1/3$ for steel failure and $c = -1/2$ for nonductile reinforced concrete.

The above equation can be written in terms of a "damage fraction" ($D = n_d / N_f$) that can be sustained for n_d cycles of loading in the damaging earthquake:

$$D = \frac{n_d}{N_f} = n_d \left(\frac{\mu_\Delta^d}{\mu_m} \right)^{-\frac{1}{c}} \qquad (5\text{-}7)$$

The remaining fatigue life then is $(1 - D)$. The displacement-based λ_D-factor can thus be defined as

$$\lambda_D = \frac{\mu_\Delta^r}{\mu_m} = (1 - D)^{-c} = \left[\frac{1}{n_d} - \left(\frac{\mu_\Delta^d}{\mu_m} \right)^{-\frac{1}{c}} \right]^{-c} \qquad (5\text{-}8)$$

In the above two equations superscripts d and r refer to the damaging earthquake and remaining life, respectively.

Thus for nonductile reinforced concrete failure taking $c = -1/2$ gives

$$\lambda_D = \sqrt{\frac{1}{n_d} - \left(\frac{\mu_\Delta}{\mu_m} \right)^2} \qquad (5\text{-}9)$$

For frictional or sliding behavior modes such as lap-splice failure of masonry infill panels, there is no limit to the displacement capability. Therefore, for these two behavior modes, $\lambda_D = 1$ at all times.

Although specific research on infill components is less developed, it is reasonable to assume that similar trends would be observed.
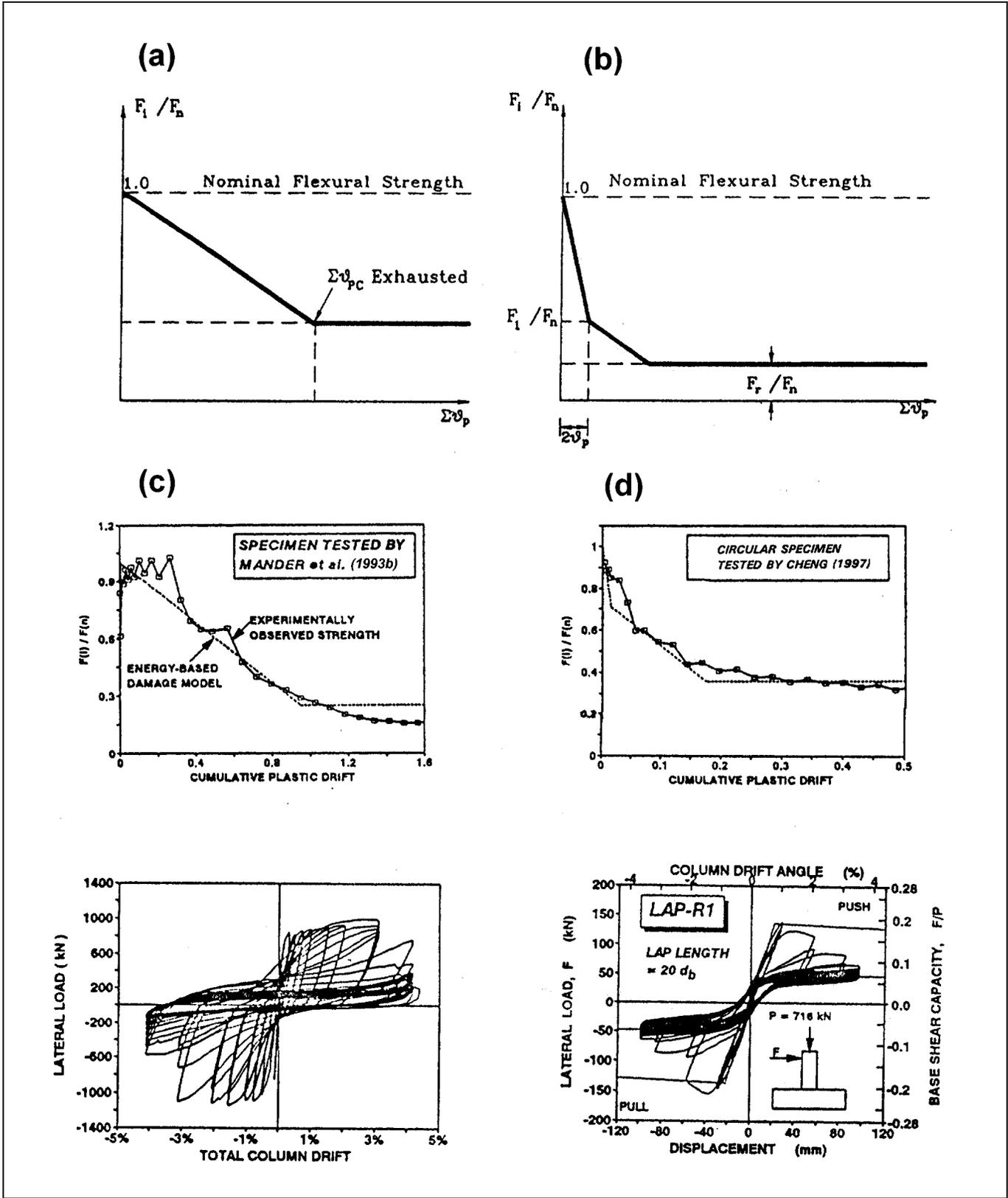
Figure 5-1 *Energy-based damage analysis of strength reduction to define* λ_Q

5.2 Tabular Bibliography for Infilled Frames

Table 5-1   Tabular Bibliography for Infilled Frames

References	A	B	C	D	E	F	G	H	I	Remarks
	Categories*									
Abrams, 1994								✓		
Al-Chaar et al., 1994								✓		
Aycardi et al., 1992									✓	Nonductile concrete frame performance
Aycardi et al., 1994									✓	Nonductile concrete frame performance
Axely and Bertero, 1979	✓							✓		Experiments on multistory frames
Benjamin and Williams, 1958		✓				✓		✓		Classic brick infilled steel frame experiments
Bertero and Brokken, 1983		✓	✓					✓		
Bracci et al., 1995									✓	Emphasis on nonductile frame performance
Brokken and Bertero, 1981								✓		
Coul, 1966						✓				
Crisafully et al., 1995		✓								
Dawe and McBride, 1985		✓				✓		✓		Steel frame with pierced brick infills
Dhanasekar et al., 1985		✓								
Flanagan and Bennett, 1994								✓		Steel frame-clay tile infill
Focardi and Manzini, 1984		✓								
Gergely et al., 1993				✓						Steel frame-clay tile infill
Hamburger and Chakradeo, 1993		✓								
Hill, 1994		✓								
Holmes, 1961						✓				
Kadir, 1974	✓	✓	✓					✓		
Kahn and Hanson, 1977				✓						
Klingner and Bertero, 1976								✓		Multistory infilled frame performance
Klingner and Bertero, 1978								✓		Multistory infilled frame performance
Kodur et al., 1995		✓								
Liauw and Lee, 1977	✓	✓				✓				
Liauw, 1979	✓							✓		Multistory steel frames-concrete infills
Liauw and Kwan, 1983a	✓							✓		Steel frame-concrete infill plastic failure modes
Liauw and Kwan, 1983b	✓	✓								Plastic-strength theory
Maghaddam and Dowling, 1987	✓	✓	✓	✓	✓	✓	✓	✓		General treatise on infilled-frame behavior
Mainstone and Weeks, 1970			✓					✓		

*A = Modes of Failure, B = Strength, C = Stiffness, D = Ductility, E = Hysteretic Performance, F = Openings, G = Repairs,
H = Experimental Performance of Infilled Frames, I = Steel and Concrete Frame Behavior

Table 5-1　　*Tabular Bibliography for Infilled Frames (continued)*

References	A	B	C	D	E	F	G	H	I	Remarks
Mainstone, 1971		✓	✓	✓				✓		Classical work on strut methods of analysis
Mallick and Garg, 1971					✓					
Mander and Nair, 1993a	✓				✓			✓		Steel frames-brick infills under cyclic loading
Mander et al., 1993b	✓				✓		✓	✓		Effect of ferrocement repairs
Mander et al., 1994									✓	Low-cycle fatigue of steel frame connections
Mander et al., 1995								✓		
Mehrabi et al., 1996								✓		Concrete frame-block infill experiments
Mosalam et al., 1994				✓						Steel frame brick infills finite-element analysis
Parducci and Mezzi, 1980		✓			✓					
Paulay and Priestley, 1992	✓	✓	✓						✓	Classical text on design
Polyakov, 1956	✓	✓	✓					✓		Earliest work on infills translated from Russian
Prawel and Lee, 1994							✓			Ferrocement repairs for masonry
Priestley, 1996									✓	Most recent work on RC in shear
Priestley et al., 1996									✓	Most recent work on RC in shear
Reinhorn et al., 1995					✓					Advanced analysis methods for infills
Riddington and Stafford-Smith, 1977	✓	✓	✓							Early work on strut methods of analysis
Riddington, 1984		✓						✓		Emphasis on gap effects
Sachanski, 1960								✓		
Saneinejad and Hobbs, 1995	✓	✓	✓							Most up-to-date reference on analysis methods
Shapiro et al., 1994								✓		
Shen and Zhu, 1994								✓		Pseudo-dynamic tests
Shing et al., 1994								✓		
Stafford-Smith, 1966		✓	✓					✓		Early experimental work
Stafford-Smith and Carter, 1969			✓							Pioneering work on analysis using strut methods
Thomas, 1953								✓		Emphasis on brick work
Wood, 1978	✓	✓								Early work on plastic methods of analysis
Yoshimura and Kikuchi, 1995								✓		
Zarnic and Tomazevic, 1984				✓	✓			✓		
Zarnic and Tomazevic, 1985a				✓	✓			✓		
Zarnic and Tomazevic, 1985b		✓		✓	✓			✓		

*A = Modes of Failure, B = Strength, C = Stiffness, D = Ductility, E = Hysteretic Performance, F = Openings, G = Repairs,
H = Experimental Performance of Infilled Frames, I = Steel and Concrete Frame Behavior

5.3   References for Infilled Frames

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