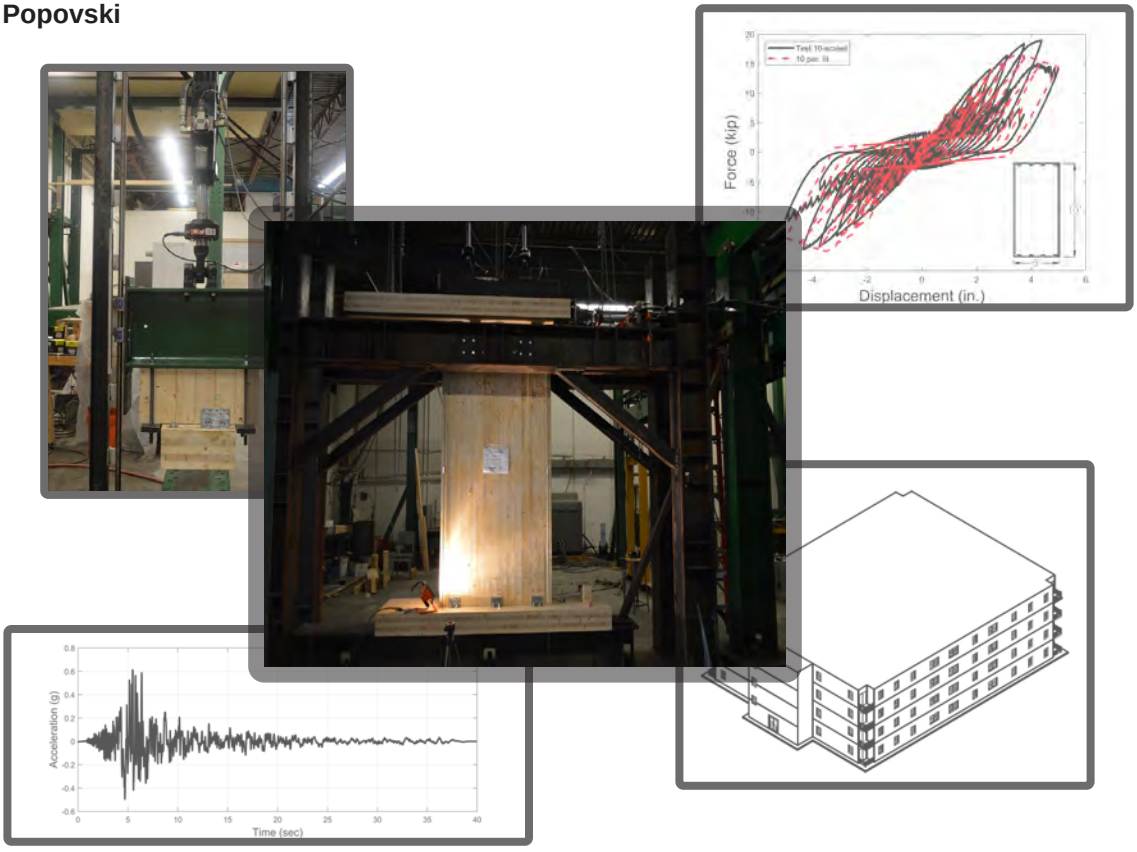
United States Department of Agriculture

Determination of Seismic Performance Factors for Cross-Laminated Timber Shear Walls Based on FEMA P695 Methodology

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Abstract

Cross-laminated timber (CLT) construction has been gaining popularity in North America. However, CLT-based seismic force resisting systems are not recognized in current U.S. design codes, which is among the many challenges preventing widespread adoption of CLT in the United States. The purpose of this study was to investigate the seismic behavior of CLT-based shear wall systems and to determine seismic performance factors, namely, the response modification factor (*R* factor), the system overstrength factor (Ω), and the deflection amplification factor (C_d), using the FEMA P695 procedure. Nine index buildings including single-family dwellings, multifamily dwellings, and commercial (including mixed use) midrise buildings were developed, from which 72 archetypes were extracted. Testing performed at the component and subassembly levels included connector tests and isolated shear wall tests. A CLT shear wall design method was developed and used to design the archetypes, which were then assessed with nonlinear pushover analysis and incremental dynamic analysis. Based on the required collapse margin, an *R* factor of 3 is proposed for CLT shear wall systems with 2:1 or mixed aspect ratio panels up to 4:1, and an *R* factor of 4 is proposed for CLT shear wall systems made up of only 4:1 aspect ratio panels. Results from this study have been proposed for recognition in U.S. building codes (such as the International Building Code) through specific change proposals to update reference standards such as ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures and Special Design Provisions for Wind and Seismic.

Keywords: cross-laminated timber, seismic design, mass timber design, FEMA P695

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Determination of Seismic Performance Factors for Cross-Laminated Timber Shear Walls Based on FEMA P695 Methodology

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Executive Summary

Cross-laminated timber (CLT) was initially introduced in Europe and has recently gained popularity in North America where it is seen as a sustainable alternative to steel and concrete in midrise construction. Although most CLT structures to date have been constructed in low seismic regions, recent tests have indicated that CLT-based lateral force resisting systems can successfully be used in regions of higher seismicity. Despite the many advantages that CLT offers, the lack of a design code and systematic design procedure is one of many challenges inhibiting widespread adoption of CLT in the United States.

The purpose of this study was to investigate the seismic behavior of CLT-based shear wall systems and to determine seismic performance factors, namely, the response modification factor (*R* factor), the system overstrength factor (Ω), and the deflection amplification factor (C_d), using the FEMA P695 procedure. The methodology is an iterative process that includes establishing design requirements, developing archetypes, performing a series of tests, developing and validating nonlinear models, nonlinear static and dynamic analysis, and evaluating performance, all in conjunction with a peer panel to provide input.

Nine index buildings including single-family dwellings, multifamily dwellings, and commercial (including mixed use) midrise buildings were developed. Archetypes were then extracted from these index buildings. Testing performed at the component and subassembly levels included connector tests and isolated shear wall tests. A critical aspect of this study is the use of generic connectors for which properties are already addressed by a design specification to facilitate building code recognition. An experimentally based performance for these generic connectors is reported as part of this study to facilitate evaluation of proprietary alternatives for seismic equivalence.

Connector tests were performed on angle brackets, used for attachment of the wall to the supporting element, and interpanel connectors. These tests showed connector thickness to be important in achieving the desired ductile behavior with lesser thickness (12 gauge) being the more favorable. Quasistatic cyclic tests were conducted for a portfolio of CLT shear walls to systematically investigate the effects of various parameters. CLT demonstrated rigid behavior with energy dissipation concentrated in the connectors. Boundary constraints and gravity loading were both found to have a beneficial effect on wall performance, i.e., higher strength and deformation capacity. Specific gravity also had a significant effect on wall behavior, whereas CLT thickness was less influential. High aspect ratio panels (4:1) demonstrated lower stiffness and substantially larger deformation capacity compared with moderate aspect ratio panels (2:1). However, based on the test results, there is probably a lower bound for aspect ratio (at 2:1) at which it ceases to benefit deformation capacity of the wall. Multipanel configurations comprised of high aspect ratio panels connected through vertical joints demonstrated considerably larger deformation capacity.

A CLT shear wall design method was developed and refined based on the test results. Phenomenological models were used in modeling CLT shear walls. The archetypes were designed based on the proposed design method and were

numerically evaluated by assessing their performance using nonlinear static and dynamic analyses. Based on the rigorous process, $R = 3$ can be used for CLT shear wall systems made up of panels with a 2:1 aspect ratio or mixed 2:1 with 4:1 and $R = 4$ can be used for cases of high aspect ratio panels only, i.e., 4:1. Results from the study will be proposed for implementation in U.S. building codes (such as the International Building Code) and its reference standards such as the American Society of Civil Engineers (ASCE) 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 2016) and Special Design Provisions for Wind and Seismic (AWC 2015b).

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Chapter 1
Introduction

1.1 Background

Wood is a versatile construction material that has been used for millennia and still is the most widely used building material in the world (Youngs 2009, FPL 2010). Wood in North America has traditionally been used in single-family and multifamily housing, commercial buildings, and bridges. With the recent shift toward sustainability and green building, there is an opportunity for wood to increase and expand its current use based on its positive attributes that include low embodied energy and low carbon impact, which make wood a preferred green building material (FPL 2010, Ritter and others 2011).

Wood is an orthotropic material, meaning that it has unique properties in mutually perpendicular directions that are longitudinal, radial, and tangential. Although variation in properties is common in all material, wood properties vary considerably because it is a natural material. As a result, engineered wood products (EWP) or wood-based composites were introduced to ensure uniformity and control over the range of properties, to efficiently use the available resources in a cost-effective way, and to open new opportunities for creative use of wood (FPL 2010, McKeever 1997, Youngs 2009).

Cross-laminated timber (CLT) is one of the recent developments in the wood industry. CLT panels are constructed of several layers of lumber boards stacked orthogonally and glued together. They are usually constructed in an odd number of layers that vary from three to seven and sometimes even more, depending on the engineering application. The thickness of individual lumber boards varies from 5/8 to 2 in. (15.9 to 50.8 mm) and the width varies from 2.4 to 9.5 in. (61 to 241.3 mm) (Karacabeyli and Douglas 2013). A typical CLT panel configuration is shown in Figure 1.1.

With its initial introduction in Europe in the early 1990s and subsequent entry into the building market between 2000 and 2005, CLT has now been commonly accepted as a new-generation engineered wood product that has the potential to expand the wood building market (UNECE/FAO 2017). This innovative mass timber product, sometimes referred to as X-Lam especially in Europe, offers a number of advantages such as mass production, prefabrication, speed of construction, and a sustainable, environmentally friendly, renewable construction product. Very good thermal insulation, acoustic performance, and fire ratings are some additional benefits of this system (Karacabeyli and Douglas 2013, Ceccotti 2008).

Applications of CLT vary widely and include residential buildings, industrial and commercial buildings, and bridges. However, it is the multistory application that is of prime interest to developers and has led to its increased popularity and emergence as an alternative product to steel and concrete for midrise construction. Numerous CLT buildings have been erected in Europe, Australia, and recently North America. Table 1.1 provides a list of these major CLT structures, and the nine-story Stadthaus built in London is shown in Figure 1.2.

1.2 Motivation and Need

Despite many advantages, the lack of CLT's inclusion in U.S. standards and design approach is one of the challenges inhibiting widespread adoption of CLT in North America and hinders its emergence as a competitive alternative to steel and concrete in midrise construction. In a study based on a nationwide survey of architectural firms across the United States regarding adoption of CLT, building code compatibility issues, initial cost, and lack of CLT manufacturers were identified as the most important barriers (Mallo and Espinoza 2015). Increasing interest in mass timber construction in the United States has motivated new CLT manufacturers to enter the market. These manufacturers currently include DR Johnson in Oregon, SmartLam in Montana, International Beams in Alabama, and recently Katerra in Washington.

Mohammad and others (2012) identified a multilevel strategy that includes development of a product standard, material design standard, and their subsequent adoption into the building codes. In the United States, there has been recent development on all these fronts that included publication of ANSI/APA PRG 320 (APA – The Engineered Wood Association 2012), addition of a chapter dedicated to CLT in the 2015 edition of the National Design Specification (NDS) for Wood Construction (AWC 2015a), and recognition of CLT in the 2015 International Building Code (ICC 2015).

One area that requires attention is the development of seismic performance and behavior of CLT lateral systems. In the early stages of building with CLT in Europe, CLT structures were mainly constructed in low seismic regions. Although CLT was introduced more than two decades ago, it was in the past decade that researchers started focusing on using CLT as a lateral force resisting system, and this triggered an increase in the number of studies geared toward investigating CLT system behavior and performance under cyclic and dynamic loading. Most of these studies originated in Europe and more recently in North America and Japan. These studies demonstrated that CLT systems can be effectively used as a lateral force resisting system. With the introduction of CLT to the U.S. construction market and the

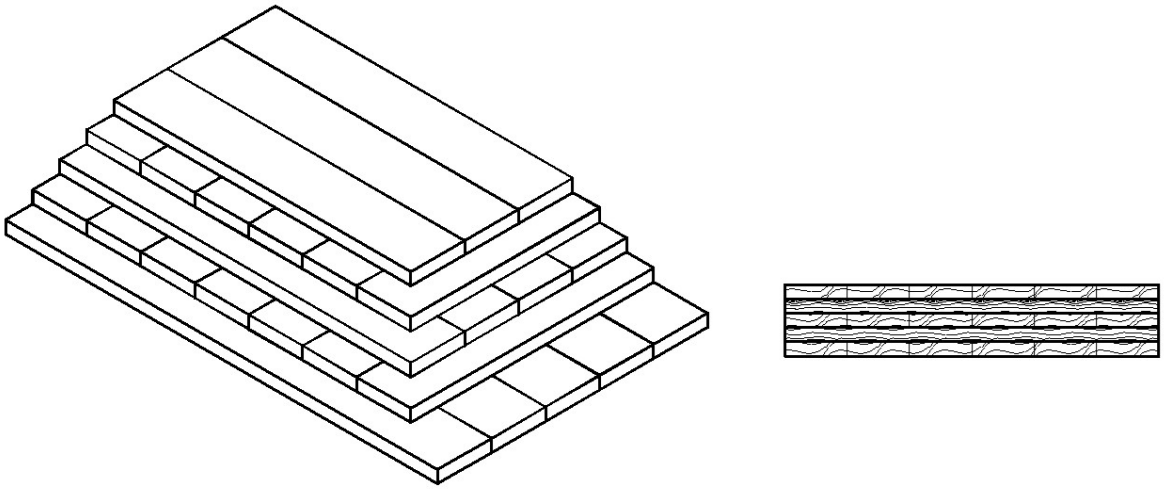


Figure 1.1. CLT panel configuration.

Table 1.1—CLT multistory structures[a]

Project	Location	Height (stories)	Comments
Stadthaus, Murray Grove[b]	London, UK	9	• Timber erected in just 9 weeks • Entire project completed in 49 weeks as opposed to 72 weeks estimated for concrete option
Bridport House	London, UK	8	
Limnologen Project	Vaxjo, Sweden	8	
Holz8 (H8)	Bad Aibling, Germany	8	
Forte[c]	Melbourne, Australia	10	• Currently the tallest CLT building in the world • CLT structure only took 10 weeks • Construction completed in 38 weeks
Cenni di Cambiamento[d]	Milan, Italy	9	• Four 9-story buildings • Project is result of competition for innovative approach to social housing
Candlewood Suites at Redstone Arsenal[e]	Huntsville, Alabama	4	• First CLT hotel in North America • 37% faster completion • 44% fewer person hours than similar hotels

[a]Forestry Innovation Investment and Binational Softwood Lumber Council (2014).
[b]Yates and others (2008).
[c]KLH Massivholz GmbH (2020).
[d]Bernasconi (2016).
[e]WoodWorks (2016).

current modern urbanization trend (Alig and others 2004), many believe that it can fill a gap for certain regions of the United States, specifically, the midrise condominium, commercial, and mixed-use building markets in seismic regions.

CLT-based seismic force resisting systems are not recognized in current U.S. design codes. CLT shear walls cannot be designed via the equivalent lateral force (ELF) design procedures (ASCE 2016). Therefore, use of CLT for seismic force resistance can only be accomplished through alternative methods. This approach, however, is usually more costly, making CLT less competitive compared with other conventional structural systems.

1.3 Research Purpose and Objective

The purpose of this study was to investigate the seismic behavior of CLT-based shear wall systems and to determine seismic performance factors for the ELF procedure. This study follows the Federal Emergency Management Agency (FEMA) P695 (FEMA 2009) methodology, which is a systematic approach that incorporates uncertainties, includes peer oversight, and integrates the design method, experimental results, and nonlinear static and dynamic analyses.

One important aspect of this study was the use of nonproprietary components and connectors already addressed by U.S. design codes to facilitate building





Figure 1.2. Stadthaus, a nine-story residential building in Hackney, London. It is the second tallest timber residential structure in the world (both photos used with permission from Waugh Thistleton Architects, London, https://creativecommons.org/licenses/bysa/4.0/legalcode).

code recognition. Test-based performance for these generic connectors is reported as part of this study to facilitate evaluation of proprietary alternatives for seismic equivalence.

The objectives of this study were the following:

- Develop and refine a design methodology for the proposed CLT system based on the applicable codes and standards
- Complete a supporting experimental investigation of CLT shear walls at the component and subassemblyw levels
- Calibrate a wall-level numerical model for archetype modeling
- Develop a number of archetypes to adequately represent the CLT design space in the United States
- Perform static and dynamic analyses on the archetypes and evaluate performance
- Provide an evaluation of seismic performance factors for CLT-based shear wall systems

1.4 Peer Review Panel

An independent team of experts is required to evaluate and comment on the approach taken by the project team, and this is an integral part in every step of the FEMA P695 methodology. The peer review panel must perform considerable judgment and provide an unbiased assessment of the development process that includes system definition with the range of application, development and/or refinement of the numerical model, archetype development, scope and extent of testing and analysis, and final selection of the seismic performance factors.

The peer review panel members for this project meet all the criteria specified in the FEMA P695 methodology and are highly qualified to critically evaluate all aspects of this project. A list of peer review panel members along with their expertise is provided below:

Member	Expertise	Role
Charlie Kircher, Ph.D., P.E. Principal and Owner, Kircher Charles & Associates	Structural and earthquake engineering, focusing on vulnerability assessment, risk analysis, and innovative design solutions	Panel chair
J. Daniel Dolan, Ph.D., P.E. Professor, Department of Civil and Environmental Engineering, Washington State University	Dynamic response of light-frame buildings, full-scale static, cyclic, and dynamic testing of structural assemblies, numerical modeling of structural and material response to static and dynamic loading	Panel member
Kelly Cobeen, S.E. Associate Principal, Wiss, Janney, and Elstner Associates, Inc.	Wood seismic design and detailing, seismic performance evaluation, structural evaluation earthquake engineering	Panel member

1.5 Report Layout

Various phases of the project consisted of development of the archetypes, design methodology, testing, modeling, and analyses. This document is organized as follows:

- Chapter 2: This chapter provides an overview of some of the major studies undertaken in Europe, North America, and Japan. As one might expect with Europe being the birthplace of CLT, most of the seismic studies originated in Europe, with two main projects being the SOFIE project in Italy and the research study in Slovenia/Macedonia.
- Chapter 3: Development of the index buildings and subsequently the resulting archetypes are presented. A total of nine index buildings including single-family, multifamily, and commercial configurations were developed, and a logical way to extract the two-dimensional archetype from these index buildings is presented.
- Chapter 4: The testing part of this project including connector testing and CLT shear wall testing is presented. Testing configurations and results are discussed for each of the phases. Connector tests include shear and uplift tests on angle brackets and shear tests on interpanel connectors. CLT shear wall tests along with the parameters that influence their behavior are discussed.
- Chapter 5: Design requirements for CLT shear wall system are presented. This includes connector design, CLT wall design, and system level requirements.
- Chapter 6: CLT archetypes developed in Chapter 3 are evaluated based on FEMA P695 methodology. Static pushover and dynamic analyses are discussed.
- Chapter 7: Collapse criteria are defined based on which collapse fragilities were developed, and archetypes are then evaluated. Uncertainties inherent in design requirements, test data, modeling methods, and variation in the ground motion records in the context of this study as part of the FEMA P695 methodology are discussed.
- Chapter 8: Finally, seismic performance factors for CLT special shear walls are proposed.

Chapter 2
Review of CLT as a Lateral Force Resisting System

The increasing interest in CLT has resulted in many studies focused on various aspects of the lateral force resisting system, which ranges from material to component to system level performance. This chapter provides an overview of some of the studies that have adopted a systematic approach to investigating seismic behavior of CLT with the eventual goal of obtaining seismic performance factors or codification of some kind.

A comprehensive research program to investigate the behavior of two-dimensional CLT wall panels was undertaken at the University of Ljubljana (Ljubljana, Slovenia) and was partially supported by KLH Massivholz GmbH (Teufenbach-Katsch, Austria) (Dujic and others 2005, 2006a, 2006b; Dujic and Zarnic 2006; KLH Massivholz GmbH 2020). The purpose of that research was to study performance of CLT panels subjected to constant vertical load combined with either monotonic or cyclic loading. Influence of various parameters such as boundary conditions, magnitude of the vertical load, and type of anchoring system were evaluated. Three cases of boundary conditions included Case A in which the top of the CLT wall was allowed to translate and rotate, Case B in which the top of the CLT wall was allowed translation with constrained rotation, and Case C in which only horizontal translation was allowed. Wall deformation response varied from cantilever to pure shear depending on the panel stiffness, magnitude of vertical load, and anchors. The results showed importance of boundary conditions on overall panel behavior.

Dujic and others (2006c, 2007, 2008) also performed a series of cyclic tests to determine the influence of openings on strength and stiffness of CLT panels. Two configurations of the wall with equal dimensions, one with a door and window opening and the other without openings, were considered for the testing. These are shown in Figure 2.1. Two specimens were tested for each configuration under identical boundary conditions. A finite element model of the test specimens was developed in commercially available software (SAP2000, Computers and Structures, Inc., New York, USA) and was calibrated using the test data. Calibrated models were used to conduct a parametric study that included 36 different opening configurations to determine the influence of the size and layout of the openings. The study resulted in simplified formulas describing the shear strength and stiffness relationship of a wall with an opening using the wall without any openings as the standard. The results indicated that for a wall with an opening area of up to 30% of the total wall area, the wall strength remained the same whereas the stiffness decreased by about 50%. Because of the high stiffness and load bearing capacity of the CLT panels, this study further confirmed that behavior is governed by the connecting elements.

Two full-scale shake table tests were performed at the IZIIS Laboratory (Skopje, Macedonia). The purpose was to investigate CLT panel behavior under dynamic loading and correlate the results with the quasistatic cyclic tests (Dujic and Zarnic 2006, Dujic and others 2006a, Hristovski and others 2012). The test configuration, shown in Figure 2.2, consisted of two walls of 8 ft by 8 ft 11 in. (2.44 by 2.72 m) with a floor diaphragm and two additional walls of 6 ft 3 in. by 8 ft 11 in. (1.905 by 2.72 m) provided in the lateral direction. One specimen was made of one-unit CLT walls, whereas the other one was constructed of two half-unit walls connected with half lap joints using screws. Steel angle brackets were used to connect the walls to the support base. Annular nails were used in the wall attachment, and bolts were used for the base attachment.

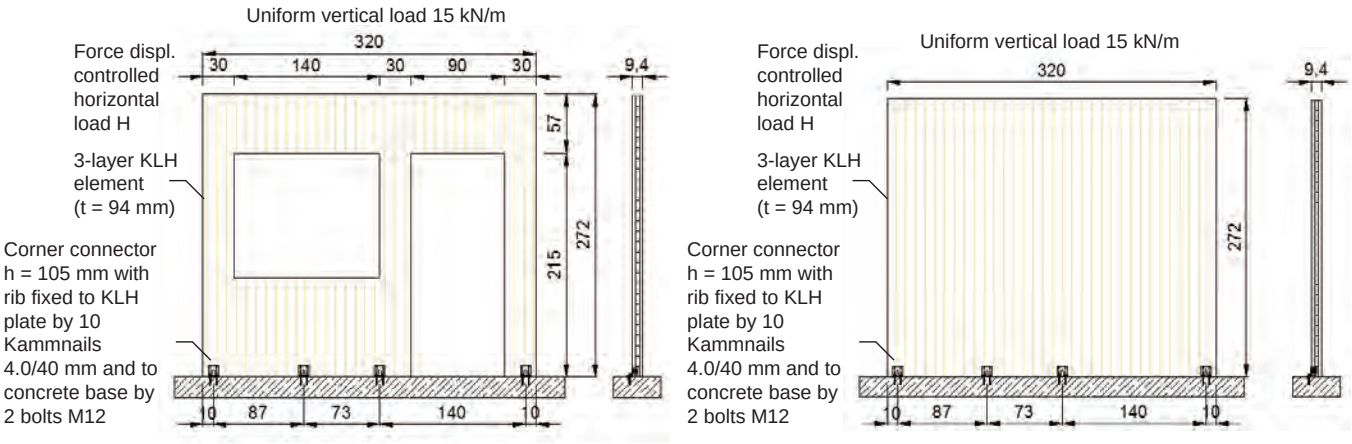


Figure 2.1. CLT wall configuration with and without openings (used with permission from·Dr. Bruno Dujic).

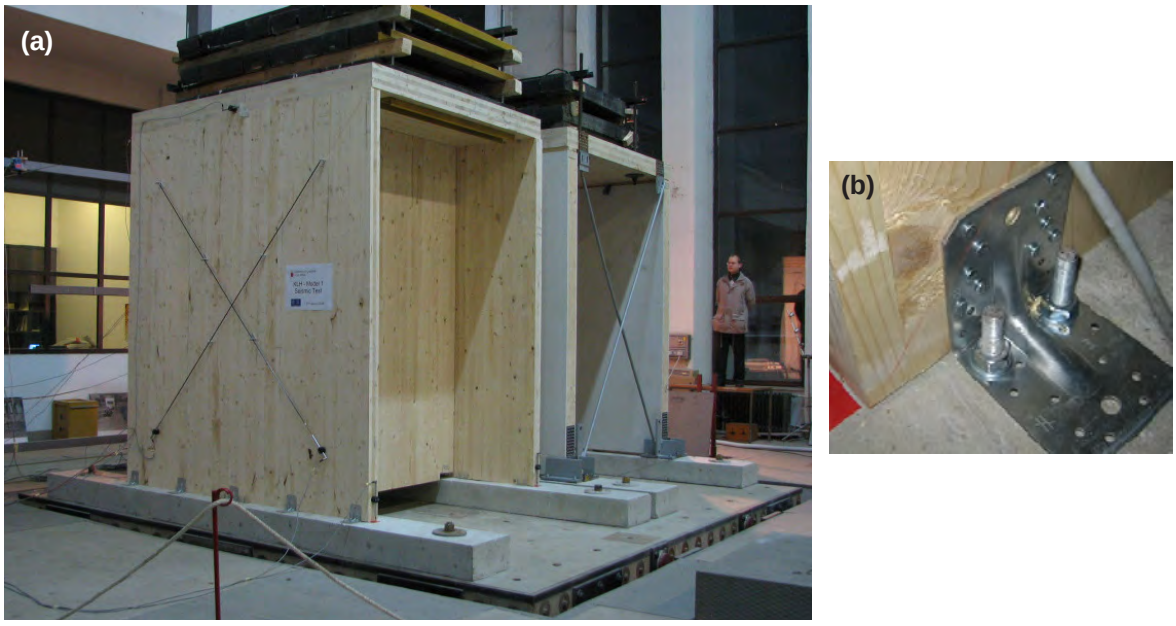


Figure 2.2. (a) Test specimen; (b) connector for base attachment (used with permission from Dr. Bruno Dujic and Otto Graf Journal).

The fundamental periods of vibration were 0.14 and 0.28 s in the longitudinal and lateral directions, respectively. Specimens were subjected to a number of ground motions, including El Centro, Petrovac, Lobe, and Friuli, and high frequency vibrations (5.0 and 7.5 Hz). Specimen 2 with the interpanel join demonstrated 42% higher relative displacement than Specimen 1. The solid wood panel exhibited linear elastic behavior, and consequently, the nonlinear behavior can be attributed to the angle bracket and vertical joint in the case of Specimen 2. The specimen with the interpanel connector exhibited more ductile behavior than the specimen with full unit walls. The system demonstrated good correlation with the quasistatic tests.

The Italian SOFIE project was a multifaceted study with the purpose of an extensive investigation of CLT behavior such as static, acoustic, thermal, and seismic performance. This collaborative effort involved the Trees and Timber Institute of the National Research Council of Italy (CRN-IVALSA), National Institute for Earth Science and Disaster Prevention (NIED) in Japan (Shizouka University), and the Building Research Institute (BRI) in Japan. The study included tests on various types of connections, quasistatic tests conducted on isolated CLT walls, pseudodynamic tests on one-story assemblies, and full-scale shake table tests on three- and seven-story buildings (Ceccotti 2008, Ceccotti and others 2010).

The results of quasistatic and psuedodynamic tests were reported by Lauriola and Sandhaas (2006). Quasistatic monotonic and cyclic tests were performed on 9 ft 8 in. by 9 ft 8 in. (2.95- by 2.95-m) CLT panels in various configurations under different vertical loadings, with and without openings. All the tests were performed using standard Simpson Stong-Tie connectors for shear and HTT22 hold-downs (Simpson Strong-Tie Company, Inc., Pleasanton, California, USA) for uplift except in a couple of cases in which custom-made hold-downs were used. The connectors are shown in Figure 2.3. Annular ringed shank nails of 0.16 by 2.36 in. (4 by 60 mm) were used to fasten the connector to the panel. Test results showed that CLT performed as rigid panels and layout and design of connections greatly influenced the wall behavior. The system exhibited very stiff yet ductile and energy dissipative behavior that makes it suitable for high seismic regions.

Pseudodynamic tests were conducted on one-story box-type structures 23 by 23 ft (7 by 7 m) in plan and 9 ft 10 in. (3 m) in height (Fig. 2.4). The specimens were tested under three different configurations that differed in terms of openings parallel to the direction of the loading. These specimens were subjected to two earthquake records, namely El Centro and Kobe JMA, scaled to two different intensities of 0.15- and 0.50-*g* peak ground acceleration (PGA). The hold-downs were designed to take the uplift, and the angle brackets were designed to take the shear loads. Based on the test results, initial stiffness for the asymmetric configuration was similar to the symmetric test, thereby confirming that wall behavior was dictated by the connectors for low magnitude shear forces.

Full-scale shake table tests on three-story CLT structures were conducted at NIED (Tsukuba, Japan) shake table facility (Ceccotti 2008; Ceccotti and others 2006a, 2006b, 2013). The three-story test specimen, shown in Figure 2.5, was 23 by 23 ft (7 by 7 m) in plan with a height of 32 ft 10 in. (10 m). Three different configurations differing in terms of the opening layout in the external walls parallel to the shaking direction were tested; Configurations A and B were symmetric, whereas Configuration C was



(a) (b)



(c)

Figure 2.3. (a) Simpson Strong-Tie BMF-105 angle bracket (now Simpson Strong-Tie ABR 105 angle bracket); (b) Simpson Strong-Tie BMF-116 angle bracket (now Simpson Strong-Tie AE116 angle bracket); (c) Simpson Strong-Tie HTT-22 hold-down (discontinued in 2010; substitute product is Simpson Strong-Tie HTT5 heavy tension tie) (used with permission from Simpson Strong-Tie Co., Inc., Pleasanton, California, USA).



Figure 2.4. Specimen for psuedodynamic testing, third configuration (used with permission from Dr. Marco Lauriola and Dr. Ario Ceccotti).



Figure 2.5. Three-story SOFIE Project building (used with permission from Dr. Ario Ceccotti).

asymmetric. A Simpson Strong-Tie HTT22 (discontinued in 2010; substitute product is Simpson Strong-Tie HTT5 heavy tension tie) was used at each wall end and opening to resist the uplift, and BMF connectors were distributed along each wall to resist the shear loads. Vertical joints were used between the walls to achieve a certain level of ductility in the system.

Each configuration was tested for three ground motions including Kobe, El Centro, and Nocera Umbra with each scaled to two different intensities of 0.15- and 0.5-*g* PGA. The specimens survived 15 destructive earthquakes without any severe damage. The asymmetric configuration of the building did not lead to any noticeable torsional effects, indicating rigid behavior of CLT panels as floor diaphragms and a counteracting contribution of perpendicular walls. It should be noted that the building was designed considering the behavior factor of $q = 1$ based on Eurocode 8 (CEN 2004), which represents elastic design without overstrength. This indicates a purely elastic design. The seismic behavior factor is defined as the ratio of PGA that causes failure to the design PGA. To systematically evaluate the *q* factor for CLT, an analytical model of the three-story building was developed in DRAIN-3DX (Regents of the University of California, Oakland, California, USA) and calibrated using the test results. The model was then subjected to a number of earthquakes, and hold-down failure was taken as the collapse mechanism. Based on the results, a *q* factor of 3 was considered reasonable (Ceccotti 2008).

The last phase of the project was a series of tri-axial shake table tests performed on a seven-story building at NIED's Miki facility in Japan (Ceccotti and others 2010, 2013). Results from previous phases of the project were used to optimize joint design for CLT buildings and to obtain ductile response. The building, shown in Figures 2.6 and 2.7, had a plan of 24 ft 7 in. by 44 ft 4 in. (7.5 by 13.5 m) and a height of 77 ft 1 in. (23.5 m). It was designed considering a *q* factor of 3 and an importance factor of 1.5 and in accordance with Eurocode 8. According to the Italian National Annex of Eurocode 8, the structure shall be designed for a PGA of 0.35 *g* corresponding to the highest hazard level. However, the structure was designed for a PGA of 0.82 *g* in the long direction and 0.60 *g* in the short direction considering the Kobe JMA earthquake. Connections were designed such that ductility and energy dissipation occurred in the hold-downs, shear connectors, and the interpanel joints. The specimen was tested only in one configuration and under three earthquakes (JMA Kobe, Nocera Umbra, and Kashiwazaki R1).

The building had an initial period of 0.43 s in the short direction (x direction) and 0.3 s in the long direction (y direction), which is quite short for a seven-story building. A maximum interstory drift of 2.64 in. (67 mm) was observed between the second and third floors, which was less than the 3.15 in. (80 mm) observed in monotonic tests. The structure remained standing throughout all the tests, and simple repairs that included tightening loose hold-down bolts and replacing connectors were performed between the tests. The system demonstrated self-centering capability and did not show significant damage up to 0.82-*g* PGA. High acceleration values up to 3.8 *g* were observed during the testing, which could be problematic for occupants and could be a future area for research. The authors concluded that, based on the test results, $q = 3$ can be taken as a reasonable value for CLT seismic design (Ceccotti and others 2013).

FPInnovations initiated CLT-related research in North America through a multidisciplinary project with the purpose of investigating seismic performance of CLT structures and more specifically to explore seismic modification factors (R_o and R_d factors). Popovski and others (2010) conducted a total of 32 monotonic and cyclic tests in 12 different configurations that consisted of different aspect ratio panels, different openings, walls with interpanel connectors, and two-story assemblies. CLT connectors included off-the-shelf steel brackets that are common for CLT applications in Europe as well as custom-made brackets (Fig. 2.8).

Results of these quasistatic tests verified rigid behavior of the CLT panels and showed that most of the deformation occurred in the steel brackets and interpanel connectors. The amount of gravity load had a greater influence on wall stiffness than did the strength of the wall. A comparison of the no-vertical-load case with that of 1.37 kip/ft (20 kN/m) load showed a 10% increase in resistance and 28% increase in stiffness in the latter case (Fig. 2.9).

Nails and screws with steel brackets gave CLT panels adequate seismic performance, and in both cases, the walls reached similar maximum lateral resistance. However, in the latter case, the capacity dropped faster at larger deformations. Hold-downs improved seismic performance, and step joints (multipanel configuration) improved ductility.

Popovski and Karacabeyli (2012) then used these tests results to develop AC130 (ICC-ES 2009) seismic equivalency in an attempt to quantify seismic performance factors for CLT in the National Building Code of Canada (NBCC). Considering the existing timber system in NBCC and recommended *q* factor in European CLT research, $R_o = 1.5$ and $R_d = 2.0$ were proposed for the CLT system. The product of these two factors is essentially the same as the *R* factor in American Society of Civil Engineers (ASCE) 7 in the United States (ASCE 2016). The results obtained from these quasistatic tests were also used by Pei and others (2013) to estimate a possible *R* factor for CLT buildings. This was achieved by investigating CLT wall behavior using a simplified kinematic model and redesigning the six-story NEESWood Capstone (van de Lindt and others 2010) building with a performance-based design procedure (PBSD). Based on the numerical analyses, an *R* factor close to 4.5 was considered reasonable for CLT

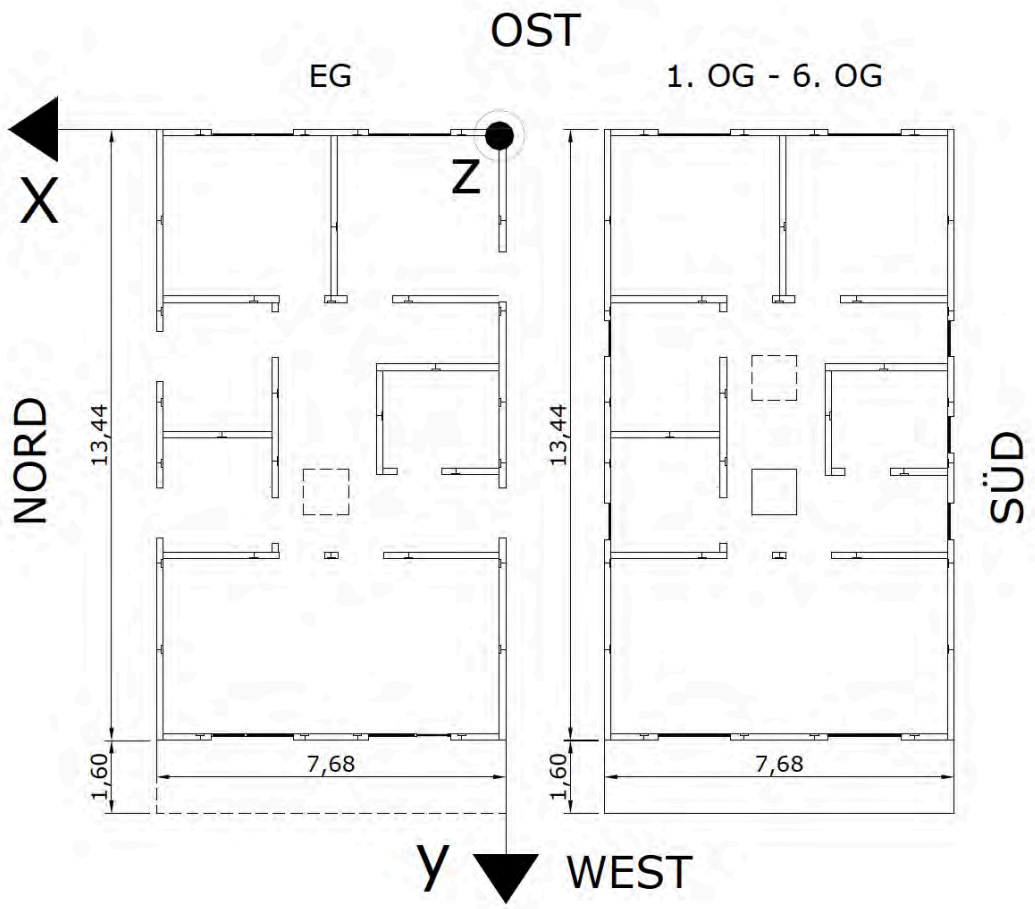


Figure 2.6. Seven-story SOFIE Project building plan (values are in units of meters) (used with permission from Dr. Ario Ceccotti).

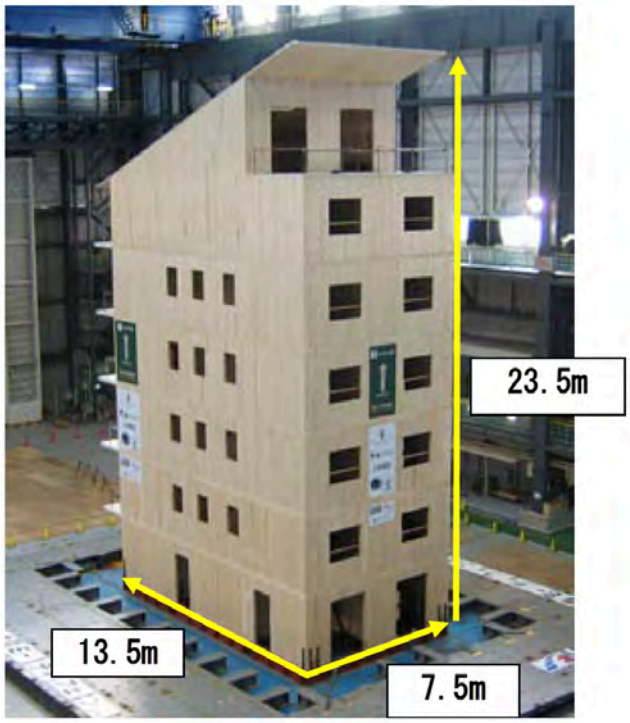


Figure 2.7. Seven-story SOFIE Project building (used with permission from Dr. Ario Ceccotti).

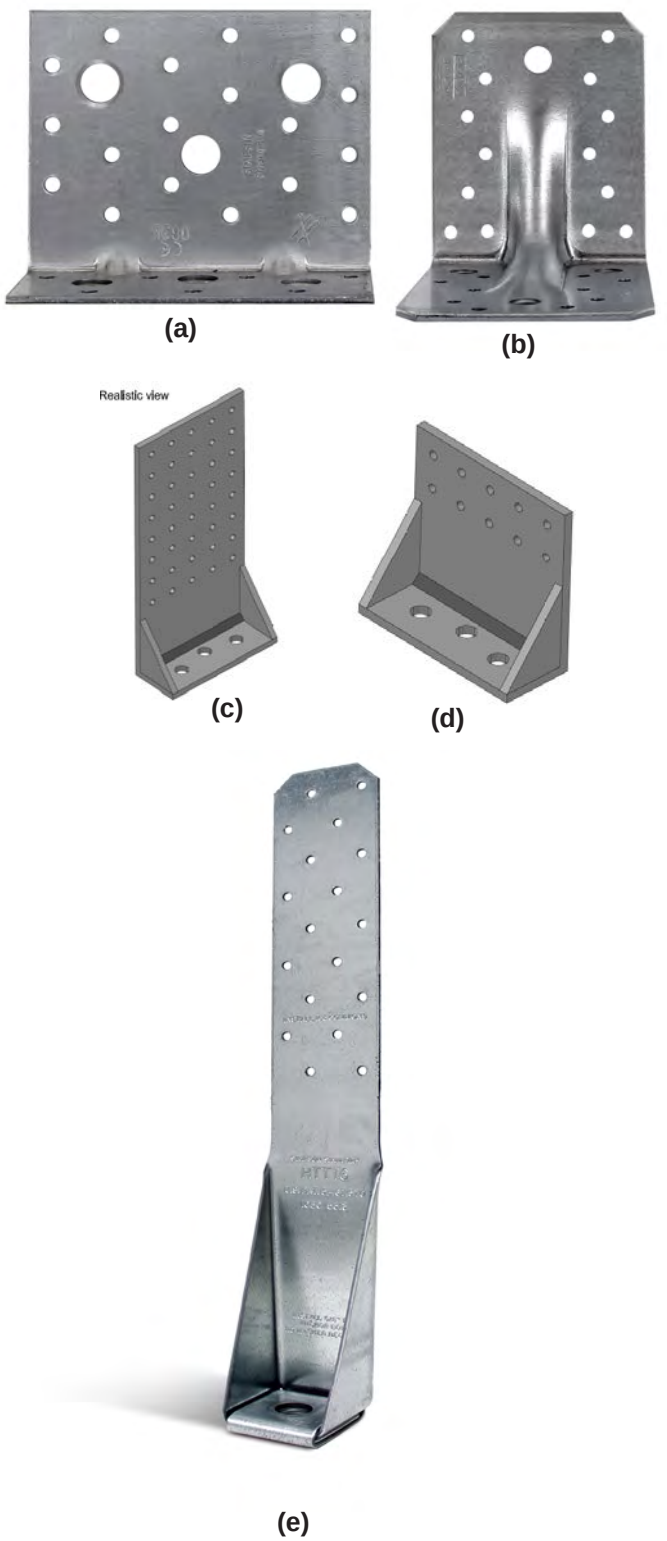


Figure 2.8. (a) Simpson Strong-Tie BMF-116 angle bracket (now Simpson Strong-Tie AE116 angle bracket); (b) Simpson Strong-Tie BMF-105 angle bracket (now Simpson Strong-Tie ABR105 angle bracket); (c) and (d) custom made, t = 0.25 in.; (e) HTT-16 hold-down (discontinued in 2010; substitute product is Simpson Strong-Tie HTT4 heavy tension tie) (a, b, and c used with permission from Simpson Strong-Tie Company, Inc. Pleasanton, California, USA; c and d used with permission from Dr. Marjan Popovski).

systems. However, the study was only performed on a single building, in a specific location, and with limited test data.

To expand on their initial finding and to better understand CLT system behavior under lateral loads, Popovski and Gavric (2015) performed a number of quasistatic monotonic and cyclic loads on a full-scale two-story structure. The structure dimensions were 19 ft 8 in. by 15 ft 9 in. (6.0 by 4.8 m) in plan with a total height of 15 ft 9 in. (4.8 m). The structure was constructed in platform-type construction and is shown in Figure 2.10. BMF brackets and HTT4 hold-downs were used to attach the wall while SFS screws were used for other connection types that included parallel wall-to-wall, perpendicular wall-to-wall, floor-to-wall, and floor-to-floor. The design was performed to ensure energy dissipation in the brackets, hold-downs, and vertical joints between parallel walls. Other joints were to incur no damage during the testing.

A total of five tests that included one pushover in the longer direction and two cyclic tests in both the longer and shorter directions of the structure were performed. To investigate the effect of additional uplift stiffening and walls perpendicular to the direction of the loading, parameters such as number of hold-downs and number of screws in perpendicular wall-to-wall connections were varied, respectively. The CLT structure performed well, exhibiting similar failure in both directions. Because of the sliding and rocking of the panels, nail shear failures in the bottom brackets of the first story walls were observed. This failure mechanism was similar in all the tests. Some torsional effects were observed in the asymmetric direction. However, similar to the findings of Lauriola and Sandhaas (2006), this did not adversely affect system behavior.

Interpanel connectors performed as expected, and floor diaphragms exhibited rigid behavior even at 3.7-in. (94-mm) thickness. Decreasing the number of screws in the vertical joint between the panels did not affect load resistance of the panel. However, it did increase deformation capacity of the structure. A maximum interstory drift of 3.2%, mostly sliding deformation, was observed during one of the tests indicating that CLT systems can accommodate larger drift requirements. Test results confirmed that walls perpendicular to the direction of the loading had a significant influence on the behavior of the building. The authors suggest that although the connectors used in this study might not be suitable for high-rise applications of CLT, they provided an insight into capacity-based design and achieving ductility in CLT buildings.

Another CLT research project was conducted at the Graz University of Technology, Austria, in collaboration with University of Kassel, Germany (Flatscher and others 2014). The testing program was divided into three steps, namely, connector tests, wall tests, and a full-scale three-story shake table testing of a CLT structure. For connector tests, a total of 215 shear and tension tests were performed

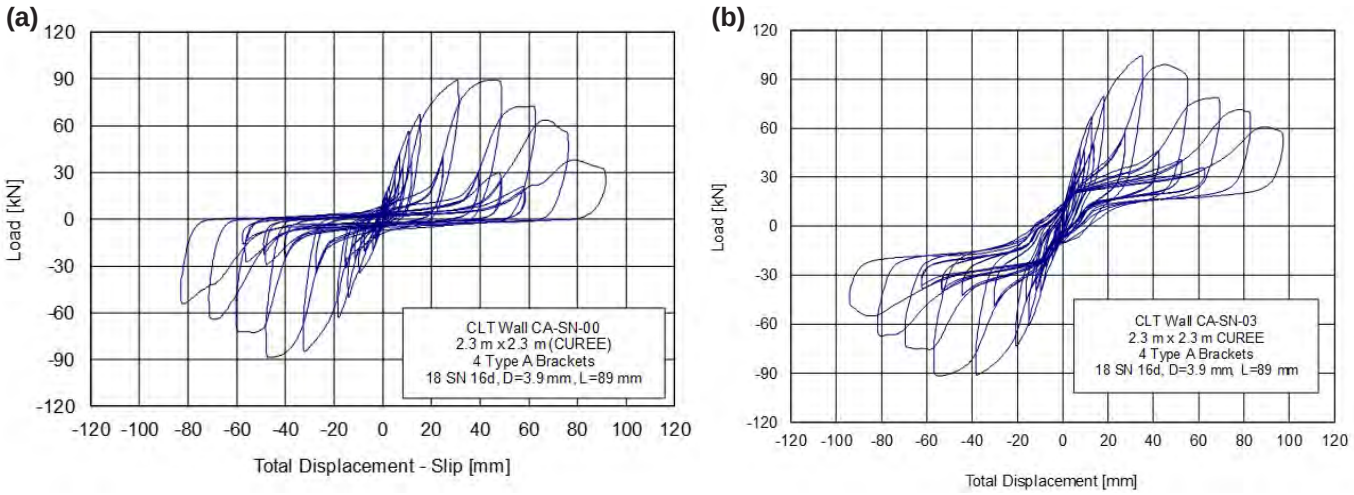


Figure 2.9. Total displacement: (a) no vertical load; (b) 1.37 kip/ft (20 kN/m) vertical load (used with permission from Dr. Marjan Popovski).

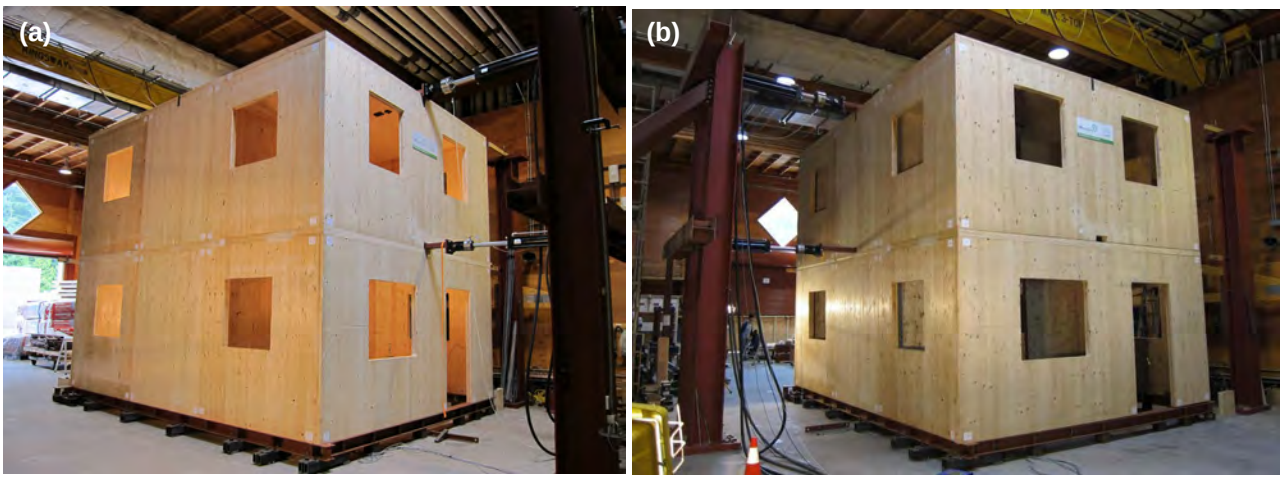


Figure 2.10. Two-story CLT structure (a) east-west direction; (b) north-south direction (used with permission from American Society of Civil Engineers (ASCE)).

on typical connectors used in CLT structures. These tests included angle brackets, hold-downs, and screws. The tests were performed in six different configurations, and each configuration was tested under monotonic and cyclic loading. Test results showed that positions of the nails and geometry of the angle bracket greatly influenced behavior of the connector.

A total of 17 wall tests were performed in five different configurations, including interpanel connectors and openings. The damping ratio of the third cycle was determined to be 15% to 20%, which was similar to the values calculated by Lauriola and Sandhaas (2006). Influence of the vertical load was also investigated as part of the study, and reduction in vertical load resulted in a decrease in stiffness, maximum strength, and ductility. Data showed that only 6% of the total deflection of the wall system was caused by shear and bending deformation, which confirmed rigid behavior of the CLT panel. However, unlike most of the previously discussed studies, test results showed that interpanel connectors did not influence wall behavior.

CLT-related research is gaining momentum in Japan in an effort to include this new proposed system in the building codes. Okabe and others (2012) conducted a number of connector tests and wall tests on specimens made from Sugi wood (Japanese cedar). The purpose was to investigate the effect of vertical load on shear capacity of Sugi CLT. Test results showed that because of gravity, stiffness increases were more noticeable than increases in strength.

Tsuchimoto and others (2014) conducted a number static and dynamic tests on the three-story CLT building shown in Figure 2.11. Shake table testing was performed for an artificial seismic wave based on the building standard law of Japan (BSL) and for the JMA Kobe record. There was no significant damage observed after the shake table testing. The results indicated that floor panels and walls perpendicular to the direction of loading had an influence on the overall response. The main conclusion of the study was



Figure 2.11. Three-story structure made of Japanese cedar CLT (used with permission from Dr. Hiroshi Isoda).

that CLT construction using tension bolts met the seismic requirements provided by BSL.

Yasumura and Ito (2014) performed a number of CLT shear wall tests considering different failure modes that included failure of the wall-to-foundation connections and interpanel connectors. The tests were conducted on walls with and without openings, and the CLT panels were made of Sugi wood. Based on the test results, precedence of the failure of the wall-to-foundation connections with no slip in the interpanel joint exhibited high capacity whereas failure of interpanel joint showed greater ductility.

In the United States, a preliminary study on the applicability of post-tensioned CLT rocking walls for tall wood building design was completed by Pei and others (2014). Experimental data were presented in Ganey and others (2017) and were subsequently used by Akbas and others (2017) to develop and calibrate numerical models. Six tests were conducted on 4- by 16-ft (1.22- by 4.88-m) rocking walls, and the design parameters of interest were PT bar size and initial stress, panel type and configuration, and supporting surface. Crushing of the CLT at the toe was observed at large lateral displacements and was the primary limit state, but large drifts in excess of 8% with the largest being 11% were shown to be possible.

Studies presented in this chapter demonstrated use of CLT as a lateral force resisting system. CLT panels exhibited rigid linear elastic behavior and deformation and energy dissipation in the connectors. Nails and screws performed well, and the hold-downs were used for overturning moment. Increase in applied gravity load resulted in increases in both stiffness and strength, albeit to a different extent, and boundary condition was observed to be an influencing parameter. These along with other findings were considered in developing the materials covered in Chapters 4 and 5, which discuss testing and design methodology, respectively.

Chapter 3
Index Buildings and Archetypes

Nine main building configurations (i.e., index building models), consisting of single-family dwellings, multifamily dwellings, and commercial (including mixed-use) midrise buildings, and 72 archetypes were considered for the purpose of this study. Floor plans are shown in Figures 3.1 through 3.9 for the nine index buildings designed as part of this research. The purpose was to verify performance of a class of building configurations but not buildings that can be considered special cases, which could potentially have unique and irregular configurations that would need to be handled on a case-by-case basis. Based on the FEMA P695 report, two-dimensional archetype wall models are considered acceptable to represent wood shear walls. Therefore, for each building in the design space, the structural design for the lateral force resisting system comes down to the selection and design of individual shear walls represented by simple archetype models. Table 3.1 lists the range of design parameters used in development of the index buildings.

Buildings between one and six stories were included in the design space, and nonstructural wall finishes were not considered part of the archetypes because they are not defined as part of the seismic force resisting system. The proposed system is only permitted under platform construction, and CLT floor diaphragms must be used for the system.

The aspect ratios specified in Table 3.1 refer to the aspect ratios of individual CLT panels. Longer walls are comprised of multiple aspect-ratio-limited CLT panels connected through interpanel connectors. For the basic wall configuration, it is envisioned that CLT walls used for seismic force resistance will be comprised of a range of 2:1 to 4:1 aspect ratio CLT panels based on the bounding values specified in the design methodology. Other features, such as gravity load and building height, which have significant influence on the system performance, will also be considered.

For this study, the proposed system was considered for seismic design category (SDC) D and the archetype designs were carried out for maximum and minimum seismic criteria of SDC D_{max} and SDC D_{min}. Maximum considered earthquake (MCE) demand is defined in terms of spectral ordinates, and design earthquake (DE) is taken as two-thirds of the MCE. Based on the methodology, site classification is taken as Site Class D, demand. Values for SDC D_{max} and SDC D_{min} are provided in Table 3.2.

The following equations are based on ASCE-7 11.4.3:

$$S_{MS} = F_a S_s \tag{3.1}$$

$$S_{M1} = F_v S_1 \tag{3.2}$$

$$S_{DS} = \tfrac{2}{3} S_{MS} \tag{3.3}$$

$$S_{D1} = \tfrac{2}{3} S_{M1} \tag{3.4}$$

where

S_{MS} and S_{M1} are MCE response spectral acceleration for short period and 1 s adjusted for site class effect, and

S_{DS} and S_{D1} are DE spectral acceleration for short period and 1 s, respectively.

Short and long period buildings are categorized based on the fundamental period and transition period equations provided in ASCE 7. If the calculated fundamental period is smaller than the transition period, T_s, the archetype is categorized as short period, and if it is larger, the archetype is categorized as long period. The equations are given as follows:

$$T = C_u T_a = C_u C_t h_n^x \tag{3.5}$$

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{S_{M1}}{S_{MS}} \tag{3.6}$$

where

T is upper limit on calculated period,

T_a is approximate fundamental period, ASCE 7 12.8.2.1,

C_u is coefficient for upper limit on calculated period, ASCE 7 table 12.8-1,

C_t and x are approximate period parameters, ASCE 7 table 12.8.2,

T_s is transition period, and

S_{D1}, S_{DS}, S_{M1}, and S_{MS} are spectral ordinates provided in Table 3.2 for SDC D_{max}.

Performance group criteria are shown in Figure 3.10. The design space was divided into various performance groups, and each one was categorized based on variables such as seismic design category, gravity load, and building height variations. Two basic configurations considered for this project were wall lengths (wall lines in a building) of 2.5 to 20 ft (0.8 to 6.1 m) and 20 to 60 ft (6.1 to 18.3 m). Other variables include the aspect ratio of the panels, gravity loads, seismic design category, and period domain. Because a number of walls can meet the criteria for a certain performance group, tributary area and the available shear wall length in each case were used as additional parameters in selection of the critical wall as an archetype. In other words, selection of shear walls from the index buildings was accomplished by extracting wall lines that had the largest tributary area to shear wall length to serve as worst-case scenarios.



Figure 3.1. Index Building 1 (1 ft = 0.3 m; 1 in. = 25.4 mm).

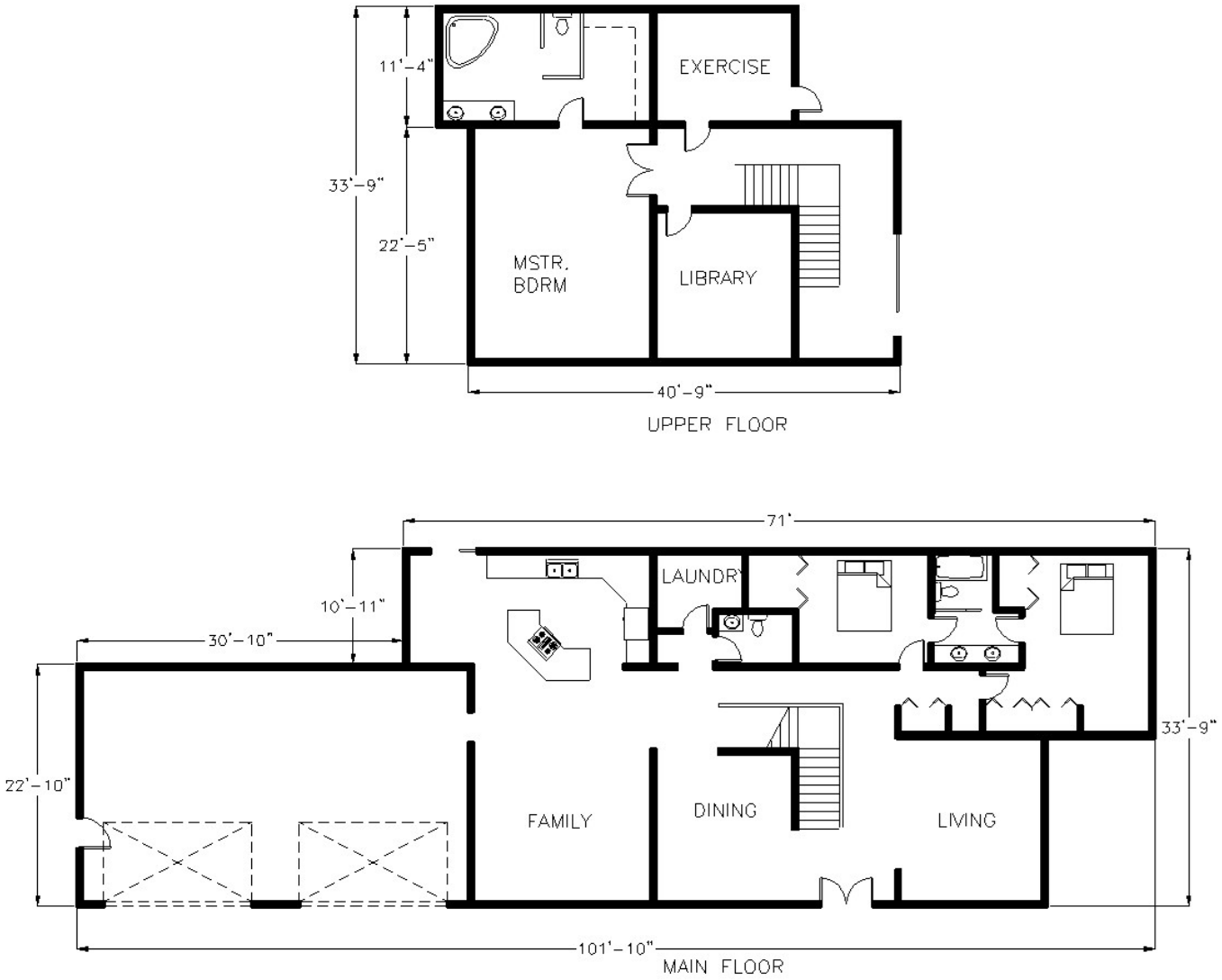


Figure 3.2. Index Building 2 (1 ft = 0.3 m; 1 in. = 25.4 mm).

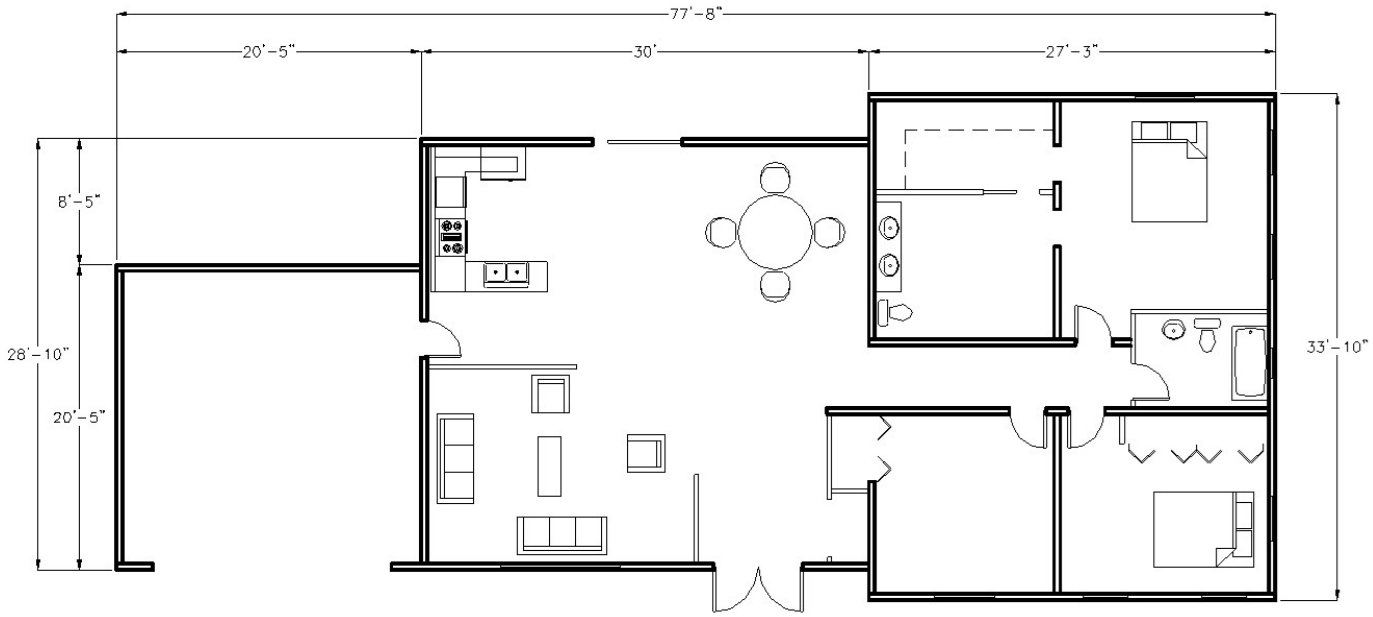


Figure 3.3. Index Building 3 (1 ft = 0.3 m; 1 in. = 25.4 mm).

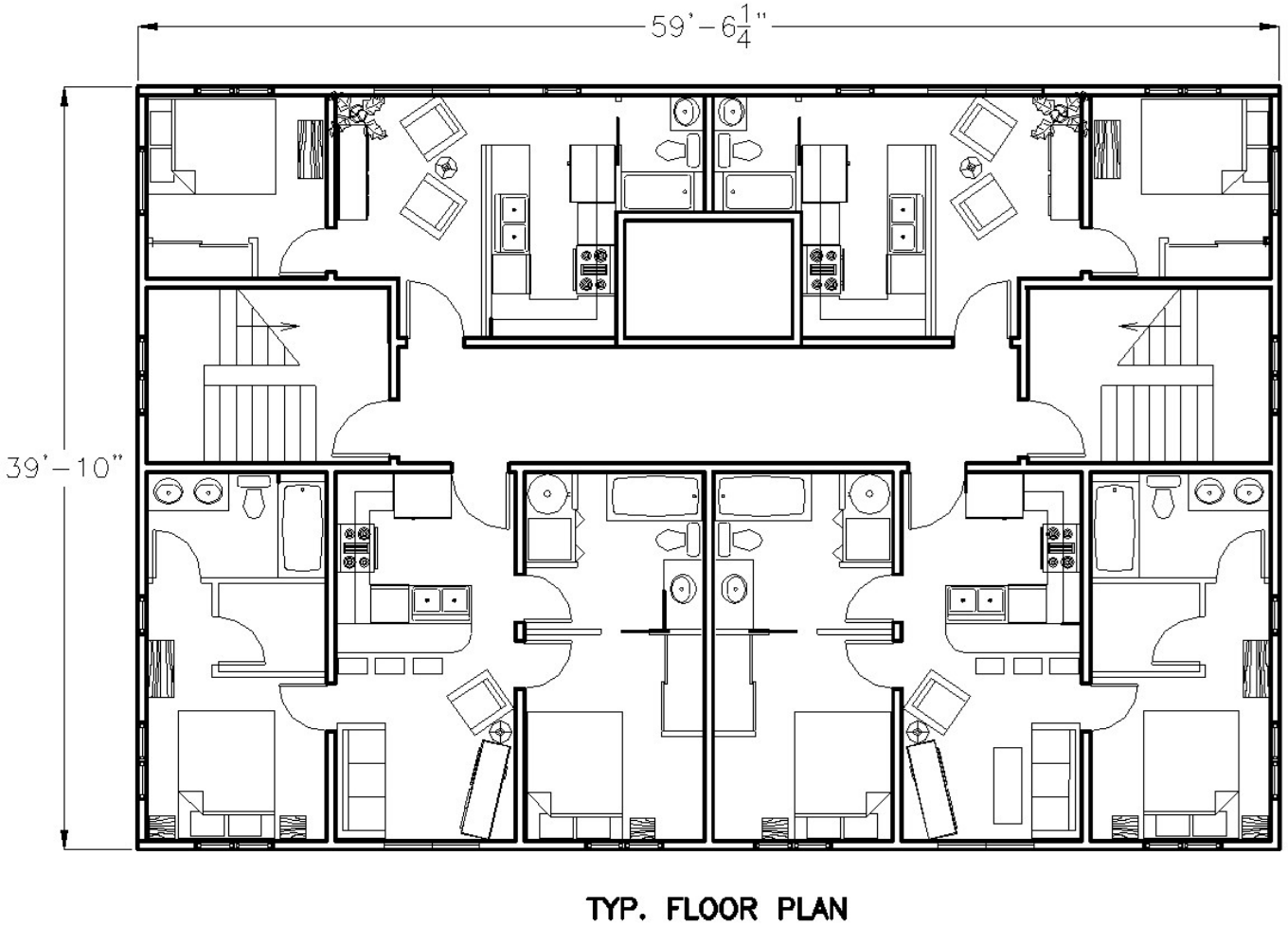


Figure 3.4. Index Building 4 (1 ft = 0.3 m; 1 in. = 25.4 mm).

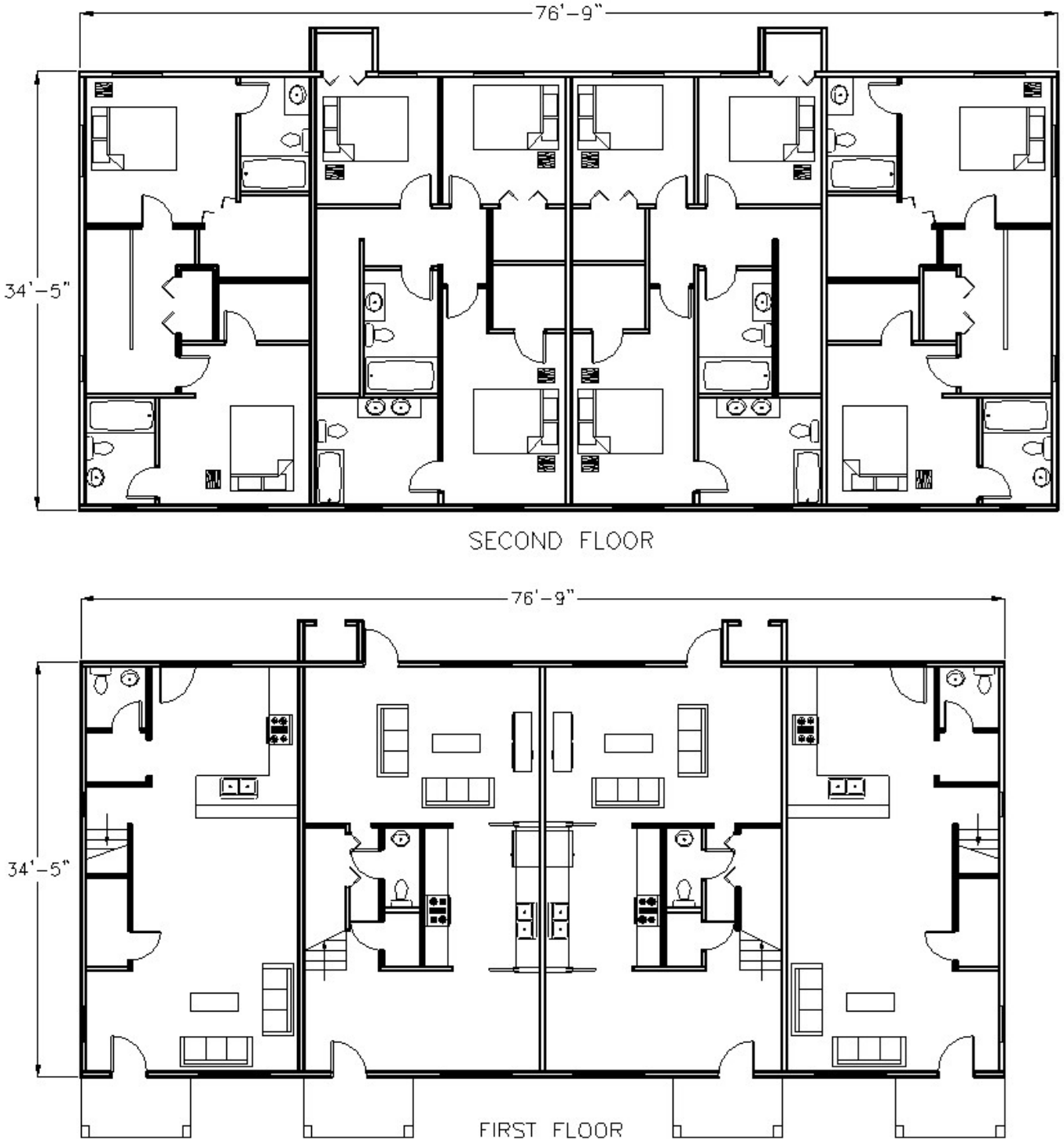


Figure 3.5. Index Building 5 (1 ft = 0.3 m; 1 in. = 25.4 mm).

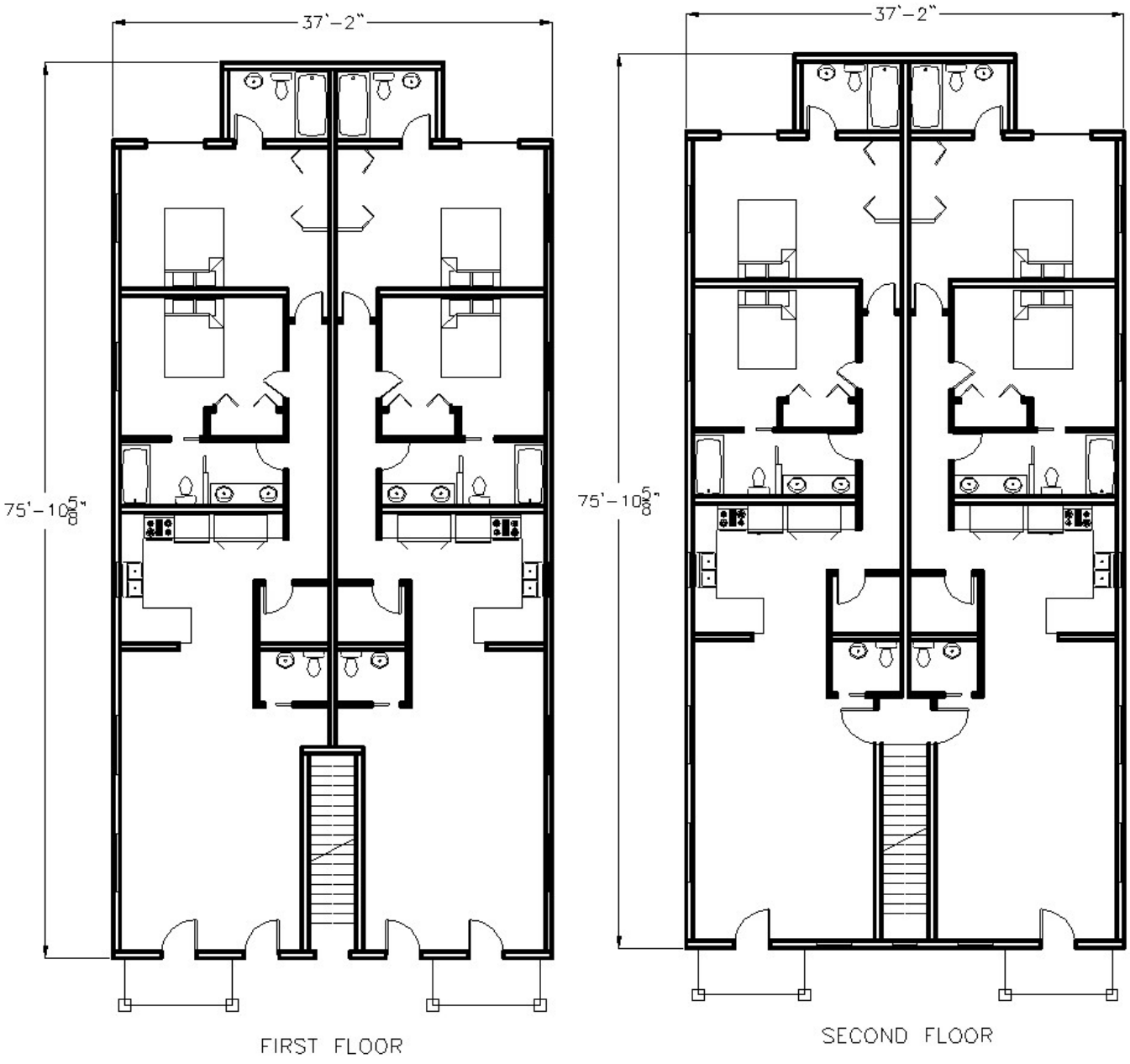


Figure 3.6. Index Building 6 (1 ft = 0.3 m; 1 in. = 25.4 mm).

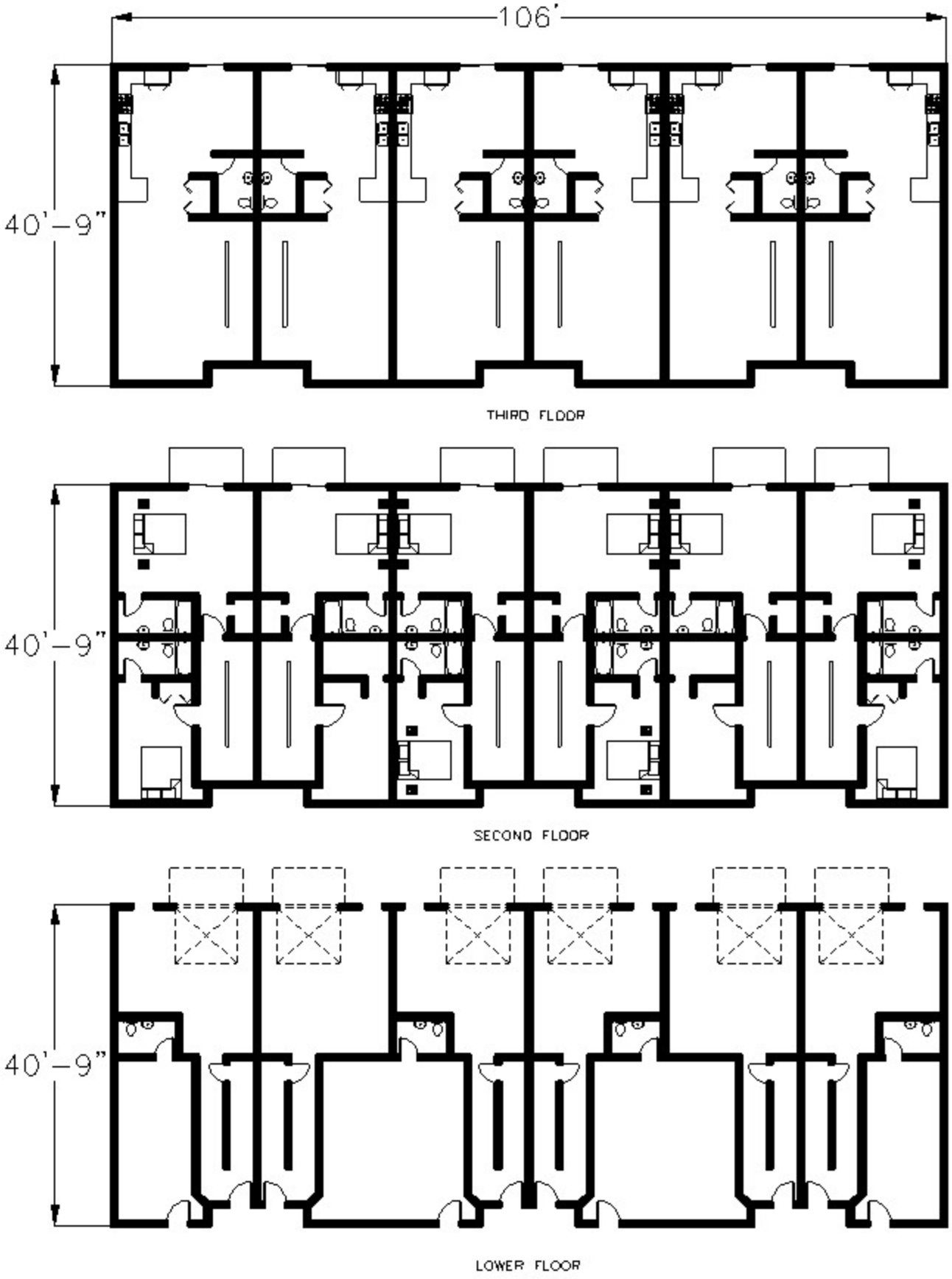


Figure 3. 7. Index Building 7 (1 ft = 0.3 m; 1 in. = 25.4 mm).

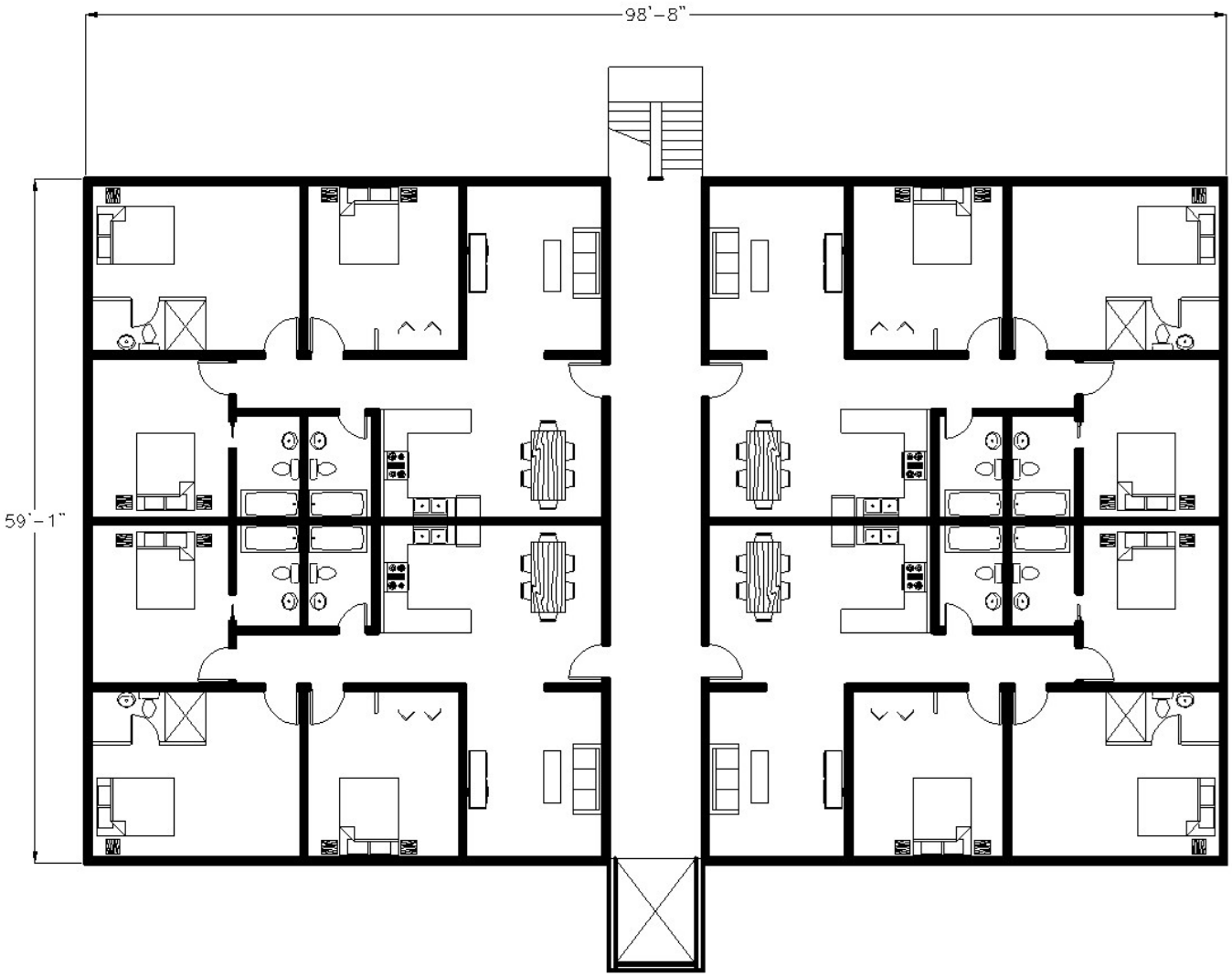


Figure 3.8. Index Building 8 (1 ft = 0.3 m; 1 in. = 25.4 mm).

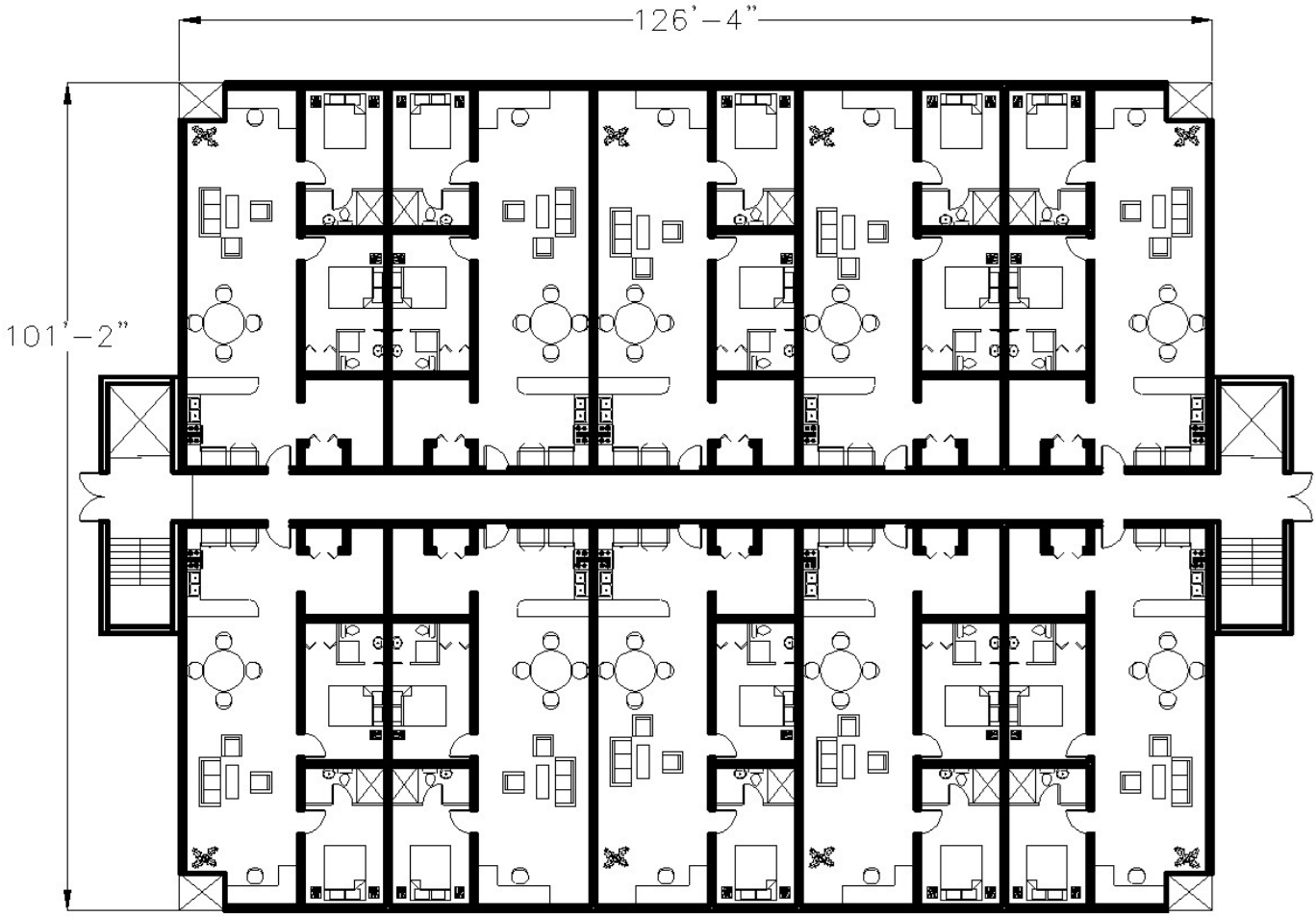


Figure 3.9. Index Building 9 (1 ft = 0.3 m; 1 in. = 25.4 mm).

Table 3.1—Range of variables considered for the definition of CLT archetypes

Variable	
Elevation and plan configuration	• Various shear wall lengths • Shear wall line of low, high, and mixed aspect ratio CLT panels
Building vertical configuration	• 1–6 stories • 10-ft story height
Interior and exterior nonstructural wall finishes	Not considered
Seismic design category	D_{max} and C_{max}/D_{min}
Gravity load	ASCE 7 table C3-1, appendix A

Table 3.2—Mapped spectral values; FEMA P695

Seismic design category	Maximum considered earthquake			Design earthquake
	Short period spectral acceleration			
	$S_s(g)$	F_a	$S_{MS}(g)$	S_{DS}
D_{max}	1.5	1.0	1.5	1.0
D_{min}	0.55	1.36	0.75	0.5
	1-s spectral acceleration			
	$S_1(g)$	F_v	$S_{M1}(g)$	S_{D1}
D_{max}	0.6	1.5	0.9	0.6
D_{min}	0.132	2.28	0.3	0.2

A number of possible walls were extracted from each index building, which were then used to design archetypes in order to populate performance groups. Table 3.3 presents a summary of the extracted walls, and the complete set of drawings for each archetype is provided in Appendix B. These walls are described as IndexBuilding_ ExtractedWallName_Number of Stories.

Looking at Table 3.3, only four walls (shown in Table 3.4) meet the criteria for 2.5- to 20-ft (0.8- to 6.1-m) wall length and are short period archetypes (their corresponding fundamental periods are less than T_s). Walls shown in Table 3.5, on the other hand, are long period (their corresponding periods are greater than T_s). Considering the available shear wall length and tributary area, the highlighted walls in Tables 3.4 and 3.5 were used in the performance groups for the 2.5- to 20-ft (0.8- to 6.1-m) wall configuration. This is shown in Table 3.6. The performance group matrix in Table 3.6 is populated with the same walls but the aspect ratio of the panels, gravity load, and SDC all vary. The archetype designation adopted here is as follows:

Index Building_Extracted Wall Name_Number of Stories_ Low_Basic Configuration_High or Mixed Aspect Ratio_ Low or High Gravity_SDC Dmax or Dmin_Short Period or Long Period

Basic configuration 2.5ft-20ft = 1, 20ft-60ft = 2; LR = Low Aspect Ratio, HR = High Aspect Ratio, MR = Mixed Aspect Ratio; LG = Low Gravity, HG = High Gravity; DX = SDC Dmax, DN = SDC Dmin; SP = Short Period, LP = Long Period

Mixed aspect ratio refers to a configuration where both high and low aspect ratio panels are used in a wall line. An example of this would be a wall line composed of 2:1 aspect ratio shear walls in line with 4:1 aspect ratio shear walls. Mixed aspect ratio does not refer to varying aspect ratio within a single multipanel shear wall. The design methodology subsequently presented specifically states that a multipanel configuration shall consist of panels with the same aspect ratios.

In the mixed aspect ratio case in Table 3.6, wall 4_B is used in lieu of wall 9_B because wall 9_B cannot be made of mixed aspect ratio configuration (see Appendix B).

Short period and long period walls with 20- to 60-ft (6.1- to 18.3-m) length are shown in Tables 3.7 and 3.8, respectively. From Table 3.7, considering the available shear wall length and the tributary area, walls 2_2, 3_A, and 5_B are critical and were used in the performance group matrix, shown in Table 3.9. Although the same walls can also be used for the high aspect ratio case, 1_A, 6_E, and 7_A were used instead because these walls can only be constructed with high aspect ratio panels. As shown in Table 3.7, both 5_B and 7_A have lengths greater than 60 ft (18.3 m). However, both have much smaller available shear wall length (<60 ft (<18.3 m)) in comparison with the total wall length, which makes these walls critical.

The highlighted walls in Table 3.8 were used in the performance group matrix shown in Table 3.9. Considering the ratio of the tributary area to the available shear wall length, 8_2 would normally be considered more critical than 4_E. However, in this case, the latter is selected to avoid having two archetypes (8_B and 8_2) from the same index building in the same performance group.

Because archetypes that are constructed with low aspect ratio panels (PG 25-32) can also be constructed in the mixed aspect ratio configuration, these same walls are used in that option (PG 41-48) (Table 3.9).

Each performance group includes at least three index archetype configurations, and for the purpose of this study, there are a total of 48 performance groups. The number of archetypes can be quite large, and it is important to decrease the number of archetypes numerically evaluated. For this particular study, the proposed system is intended for SDC D and the system will be assessed for SDC D_{max} and SDC D_{min}. After the preliminary analysis shows that the system performs acceptably for SDC D_{min}, this will indicate that there is no need to check for SDC C and SDC B and the system will only be analyzed for SDC D_{max}. A number of examples in FEMA P695 used this approach to decrease the

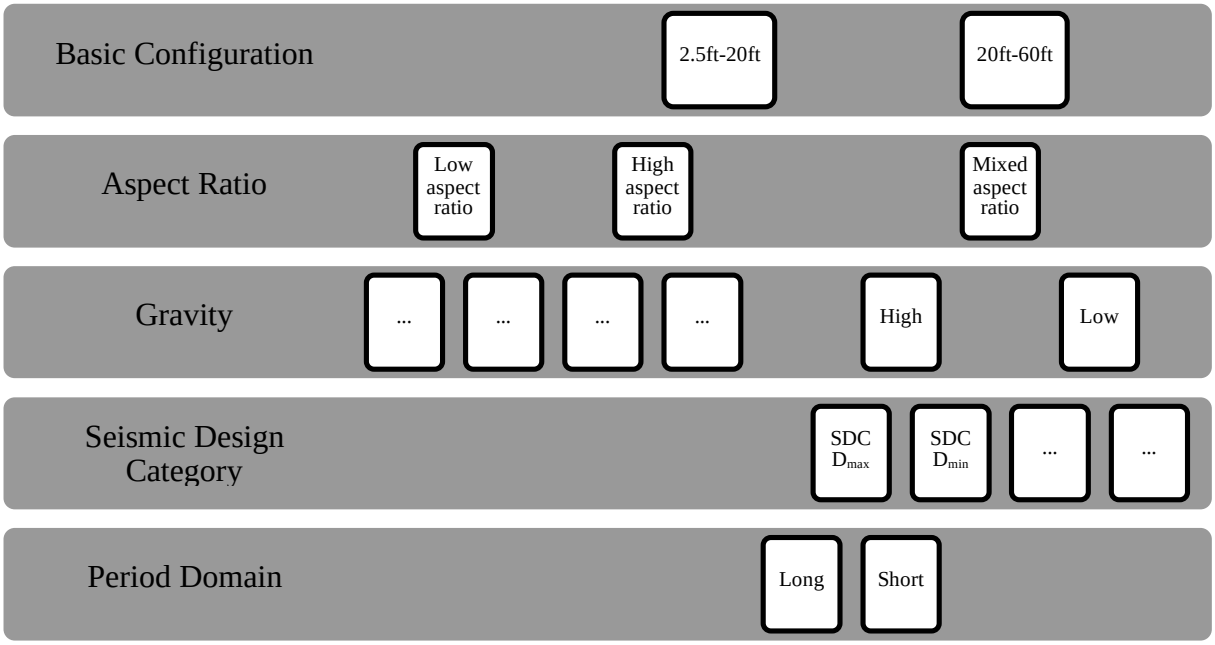


Figure 3.10. Permanence group criteria (1 ft = 0.3 m).

Table 3.3—Extracted walls (1 ft = 0.3 m; 1 ft^2 = 0.09 m^2)

Extracted wall[a]	Number of stories	Wall length (ft)	Wall opening (ft)[b]	Tributary area (ft^2)	Low aspect ratio[c]	High aspect ratio[c]
1_A	3	43	18.5	294	—	x
1_E	3	16	9.8	448	x	x
1_G	3	43	28.0	254	x	x
2_2	2	22	14.0	558	x	x
2_3	2	32.9	24.9	518	x	x
2_4	2	7.5	7.5	625	x	x
3_A	1	33	23.5	888	x	x
4_1	4,6	39.8	25.5	205	x	x
4_3	4,6	19	12.9	392	x	x
4_B	4,6	13	13.0	170	x	x
4_D	4,6	58.6	52.6	781	x	x
4_E	4,6	58.6	34.6	600	x	x
5_3	2	33.6	33.6	692	x	x
5_B	2	76.8	40.8	1,321	x	x
6_1	2	68	47.0	656	x	x
6_2	2	54	54.0	1,388	x	x
6_D	2	5	5.0	176	x	x
6_E	2	37	22.0	496	—	x
7_1	3	39.9	27.9	381	x	x
7_3	3	18.8	18.8	703	x	x
7_A	3	98.9	32.9	1,072	—	x
8_2	4,6	28.5	23.2	441	x	x
8_3	4,6	14.5	14.5	365	x	x
8_B	4,6	37.75	16.9	682	x	x
9_1	6	42.4	36.4	651	x	x
9_3	6	46.4	46.4	1,272	x	x
9_B	6	10	10.0	594	x	x

[a]Extracted wall description => IndexBuilding_ExtractedWallName.
[b]Available shear wall length = total wall length – openings for doors and windows.
[c]This indicates whether the archetype can be constructed with low aspect ratio panels (h/b = 2) or with high aspect ratio panels (h/b = 4).

Table 3.4—2.5- to 20-ft wall, short-period archetypes (1 ft = 0.3 m; 1 ft^2 = 0.09 m^2)

Extracted wall	Number of stories	Wall length (ft)	Wall opening (ft)	Tributary area (ft^2)	Low aspect ratio	High aspect ratio
1_E	3	16	9.8	448	x	x
2_4	2	7.5	7.5	625	x	x
6_D	2	5	5.0	176	x	x
7_3	3	18.8	18.8	669	x	x

Table 3.5—2.5- to 20-ft wall, long-period archetypes (1 ft = 0.3 m; 1 ft^2 = 0.09 m^2)

Extracted wall	Number of stories	Wall length (ft)	Wall opening (ft)	Tributary area (ft^2)	Low aspect ratio	High aspect ratio
4_3	4,6	19	12.9	392	x	x
4_B	4,6	13	13.0	170	x	x
8_3	4,6	14.5	14.5	365	x	x
9_B	6	10	10.0	594	x	x

number of archetypes to a manageable number. In addition, based on the testing, the critical case of the panel aspect ratio will be determined. In line with examples in FEMA P695, preliminary analysis and engineering judgment were used to further decrease the number of archetypes.

Seismic loads are defined in terms of SDC and occupancy category of the structure. Based on the methodology, structures are considered Occupancy Category I or II and they receive an importance factor equal to unity. The archetypes were designed for the DE based on the ELF procedure explained in section 12.8 of ASCE 7 and were then evaluated for the MCE. MCE demand is defined in terms of spectral ordinates presented earlier in this chapter. Preparation of index archetype designs requires selection of initial (trial) values for the response modification factor, overstrength factor, and deflection amplification factor. Trial values for R were 3 and 4 with C_d = 3. In total, nine index buildings and 72 archetypes were considered in this study.

Table 3.6—Performance group (PG) matrix for 2.5- to 20-ft (0.8- to 61-m) wall configuration

	Grouping criteria				
	Design load level				
Basic configuration	Gravity	Seismic design load	Period domain	Archetype description	Archetype No.
				PG-1	
Low aspect ratio panels	High	D_{max}	Short	1_E_3_1_LR_HG_DX_SP	1
				2_4_2_1_LR_HG_DX_SP	2
				6_D_2_1_ LR_HG_DX_SP	3
				PG-2	
			Long	4_3_ 6_1_LR_HG_ DX_LP	4
				8_3_6_1_ LR_HG_ DX_LP	5
				9_B_6 _1_ LR_HG_ DX_LP	6
				PG-3	
		D_{min}	Short	1_E_3_1_LR_HG_DN_SP	7
				2_4_2_1_LR_HG_DN_SP	8
				6_D_2_1_ LR_HG_DN_SP	9
				PG-4	
			Long	4_3_ 6_1_LR_HG_ DN_LP	10
				8_3_6_1_ LR_HG_ DN_LP	11
				9_B_6 _1_ LR_HG_ DN_LP	12
				PG-5	
	Low	D_{max}	Short	1_E_3_1_LR_LG_DX_SP	13
				2_4_2_1_LR_LG_DX_SP	14
				6_D_2_1_ LR_LG_DX_SP	15
				PG-6	
			Long	4_3_ 6_1_LR_LG_ DX_LP	16
				8_3_6_1_ LR_LG_ DX_LP	17
				9_B_6 _1_ LR_LG_ DX_LP	18
				PG-7	
		D_{min}	Short	1_E_3_1_LR_LG_DN_SP	19
				2_4_2_1_LR_LG_DN_SP	20
				6_D_2_1_ LR_LG_DN_SP	21
				PG-8	
			Long	4_3_ 6_1_LR_LG_ DN_LP	22
				8_3_6_1_ LR_LG_ DN_LP	23
				9_B_6 _1_ LR_LG_ DN_LP	24
				PG-9	
High aspect ratio panels	High	D_{max}	Short	1_E_3_1_HR_HG_DX_SP	25
				2_4_2_1_HR_HG_DX_SP	26
				6_D_2_1_HR_HG_DX_SP	27
				PG-10	
			Long	4_3_ 6_1_HR_HG_ DX_LP	28
				8_3_6_1_ HR_HG_ DX_LP	29
				9_B_6 _1_HR_HG_ DX_LP	30
				PG-11	
		D_{min}	Short	1_E_3_1_HR_HG_DN_SP	31
				2_4_2_1_HR_HG_DN_SP	32
				6_D_2_1_ HR_HG_DN_SP	33
				PG-12	
			Long	4_3_ 6_1_HR_HG_ DN_LP	34
				8_3_6_1_ HR_HG_ DN_LP	35
				9_B_6 _1_ HR_HG_ DN_LP	36

Table 3.6—Performance group (PG) matrix for 2.5- to 20-ft (0.8- to 61-m) wall configuration (con.)

Basic configuration	Grouping criteria: Design load level: Gravity	Seismic design load	Period domain	Archetype description	Archetype No.
				PG-13	
High aspect ratio panels (con.)	Low	D_{max}	Short	1_E_3_1_HR_LG_DX_SP	37
				2_4_2_1_HR_LG_DX_SP	38
				6_D_2_1_ HR_LG_DX_SP	39
				PG-14	
			Long	4_3_ 6_1_HR_LG_ DX_LP	40
				8_3_6_1_ HR_LG_ DX_LP	41
				9_B_6 _1_ HR_LG_ DX_LP	42
				PG-15	
		D_{min}	Short	1_E_3_1_HR_LG_DN_SP	43
				2_4_2_1_HR_LG_DN_SP	44
				6_D_2_1_ HR_LG_DN_SP	45
				PG-16	
			Long	4_3_ 6_1_HR_LG_ DN_LP	46
				8_3_6_1_ HR_LG_ DN_LP	47
				9_B_6 _1_ HR_LG_ DN_LP	48
				PG-17	
Mixed aspect ratio panels	High	D_{max}	Short	8_3_4_1_MR_HG_DX_SP	49
				4_3_ 4_1_MR_HG_ DX_LP	50
				7_3_3_1_MR_HG_DX_SP	51
				PG-18	
			High	4_3_ 6_1_MR_HG_ DX_LP	52
				8_3_6_1_ MR_HG_ DX_LP	53
				4_B_6_1_ MR_HG_ DX_LP	54
				PG-19	
		D_{min}	Short	8_3_4_1_MR_HG_DN_SP	55
				4_3_ 4_1_MR_HG_ DN_SP	56
				7_3_3_1_MR_HG_DN_SP	57
				PG-20	
			Long	4_3_ 6_1_MR_HG_ DN_LP	58
				8_3_6_1_ MR_HG_ DN_LP	59
				4_B_6_1_ MR_HG_ DN_LP	60
				PG-21	
	Low	D_{max}	Short	8_3_4_1_MR_LG_DX_SP	61
				4_3_ 4_1_MR_LG_ DX_SP	62
				7_3_3_1_MR_LG_DX_SP	63
				PG-22	
			Long	4_3_ 6_1_MR_LG_ DX_LP	64
				8_3_6_1_ MR_LG_ DX_LP	65
				4_B_6_1_ MR_LG_ DX_LP	66
				PG-23	
		D_{min}	Short	8_3_4_1_MR_LG_DN_SP	67
				4_3_ 4_1_MR_LG_ DN_SP	68
				7_3_3_1_MR_LG_DN_SP	69
				PG-24	
			Long	4_3_ 6_1_MR_LG_ DN_LP	70
				8_3_6_1_ MR_LG_ DN_LP	71
				4_B_6_1_ MR_LG_ DN_LP	72

Table 3.7—20- to 60-ft wall, short-period archetypes (1 ft = 0.3 m, 1 ft^2 = 0.09 m^2)

Extracted wall	Number of stories	Wall length (ft)	Wall opening (ft)	Tributary area (ft^2)	Low aspect ratio	High aspect ratio
1_A	3	43	18.5	294	—	x
1_G	3	43	28.0	254	x	x
2_2	2	22	14.0	558	x	x
2_3	2	32.9	24.9	518	x	x
3_A	1	33	23.5	888	x	x
5_3	2	33.6	33.6	692	x	x
5_B	2	76.75	40.8	1,321	x	x
6_1	2	68	47.0	656	x	x
6_2	2	54	54.0	1,388	x	x
6_E	2	37	22.0	496	—	x
7_1	3	39.9	27.9	381	x	x
7_A	3	98.9	32.9	1,042	—	x

Table 3.8—20- to 60-ft wall, long-period archetypes (1 ft = 0.3 m, 1 ft^2 = 0.09 m^2)

Extracted wall	Number of stories	Wall length (ft)	Wall opening (ft)	Tributary area (ft^2)	Low aspect ratio	High aspect ratio
4_1	4,6	39.8	25.5	205.2	x	x
4_D	4,6	58.6	52.6	781	x	x
4_E	4,6	58.6	34.6	600	x	x
8_2	4,6	28.5	23.2	440.5	x	x
8_B	4,6	37.8	16.9	682	x	x
9_1	6	42.4	36.4	650.75	x	x
9_3	6	46.4	46.4	1,272	x	x

Table 3.9—Performance group (PG) matrix for 20- to 60-ft (6.1- to 18.3-m) wall configuration

Basic configuration	Grouping criteria: Design load level: Gravity	Seismic design load	Period domain	Archetype description	Archetype No.
				PG-25	
Low aspect ratio panels	High	D_{max}	Short	2_2_2_2_ LR_HG_ DX_SP	73
				3_2_1_2_ LR_HG_ DX_SP	74
				5_B_2_2_ LR_HG_ DX_SP	75
				PG-26	
			Long	4_E_6_2_ LR_HG_ DX_LP	76
				8_B_6_2_ LR_HG_ DX_LP	77
				9_3_6_2_ LR_HG_ DX_LP	78
				PG-27	
		D_{min}	Short	2_2_2_2_ LR_HG_ DN_SP	79
				3_2_1_2_ LR_HG_ DN_SP	80
				5_B_2_2_ LR_HG_ DN_SP	81
				PG-28	
			Long	4_E_6_2_ LR_HG_ DN_LP	82
				8_B_6_2_ LR_HG_ DN_LP	83
				9_3_6_2_ LR_HG_ DN_LP	84
				PG-29	
	Low	D_{max}	Short	2_2_2_2_ LR_LG_ DX_SP	85
				3_2_1_2_ LR_LG_ DX_SP	86
				5_B_2_2_ LR_LG_ DX_SP	87
				PG-30	
			Long	4_E_6_2_ LR_LG_ DX_LP	88
				8_B_6_2_ LR_LG_ DX_LP	89
				9_3_6_2_ LR_LG_ DX_LP	90
				PG-31	
		D_{min}	Short	2_2_2_2_ LR_LG_ DN_SP	91
				3_2_1_2_ LR_LG_ DN_SP	92
				5_B_2_2_ LR_LG_ DN_SP	93
				PG-32	
			Long	4_E_6_2_ LR_LG_ DN_LP	94
				8_B_6_2_ LR_LG_ DN_LP	95
				9_3_6_2_ LR_LG_ DN_LP	96
				PG-33	
High aspect ratio panels	High	D_{max}	Short	1_A_3_2_ HR_HG_ DX_SP	97
				6_E_2_2_ HR_HG_ DX_SP	98
				7_A_3_2_ HR_HG_ DX_SP	99
				PG-34	
			Long	4_E_6_2_ HR_HG_ DX_LP	100
				8_B_6_2_ HR_HG_ DX_LP	101
				9_3_6 _2_ HR_HG_ DX_LP	102
				PG-35	
		D_{min}	Short	1_A_3_2_ HR_HG_ DN_SP	103
				6_E_2_2_ HR_HG_ DN_SP	104
				7_A_3_2_ HR_HG_ DN_SP	105
				PG-36	
			Long	4_E_6_2_ HR_HG_ DN_LP	106
				8_B_6_2_ HR_HG_ DN_LP	107
				9_3_6 _2_ HR_HG_ DN_LP	108

Table 3.9—Performance group (PG) matrix for 20- to 60-ft (6.1- to 18.3-m) wall configuration (con.)

	Grouping criteria				
	Design load level				
Basic configuration	Gravity	Seismic design load	Period domain	Archetype description	Archetype No.
				PG-37	
High aspect ratio panels (con.)	Low	D_{max}	Short	1_A_3_2_ HR_LG_ DX_SP	109
				6_E_2_2_ HR_LG_ DX_SP	110
				7_A_3_2_ HR_LG_ DX_SP	111
				PG-38	
			Long	4_E_6_2_ HR_LG_ DX_LP	112
				8_B_6_2_ HR_LG_ DX_LP	113
				9_3_6 _2_ HR_LG_ DX_LP	114
				PG-39	
		D_{min}	Short	1_A_3_2_ HR_LG_ DN_SP	115
				6_E_2_2_ HR_LG_ DN_SP	116
				7_A_3_2_ HR_LG_ DN_SP	117
				PG-40	
			Long	4_E_6_2_ HR_LG_ DN_LP	118
				8_B_6_2_ HR_LG_ DN_LP	119
				9_3_6_2_ HR_LG_ DN_LP	120
				PG-41	
Mixed aspect ratio panels	High	D_{max}	Short	2_2_2_2_ MR_HG_ DX_SP	121
				3_2_1_2_ MR_HG_ DX_SP	122
				5_B_2_2_ MR_HG_ DX_SP	123
				PG-42	
			High	4_E_6_2_ MR_HG_ DX_LP	124
				8_B_6_2_ MR_HG_ DX_LP	125
				9_3_6_2_ MR_HG_ DX_LP	126
				PG-43	
		D_{min}	Short	2_2_2_2_ MR_HG_ DN_SP	127
				3_2_1_2_ MR_HG_ DN_SP	128
				5_B_2_2_ MR_HG_ DN_SP	129
				PG-44	
			Long	4_E_6_2_ MR_HG_ DN_LP	130
				8_B_6_2_ MR_HG_ DN_LP	131
				9_3_6_2_ MR_HG_ DN_LP	132
				PG-45	
	Low	D_{max}	Short	2_2_2_2_ MR_LG_ DX_SP	133
				3_2_1_2_ MR_LG_ DX_SP	134
				5_B_2_2_ MR_LG_ DX_SP	135
				PG-46	
			Long	4_E_6_2_ MR_LG_ DX_LP	136
				8_B_6_2_ MR_LG_ DX_LP	137
				9_3_6_2_ MR_LG_ DX_LP	138
				PG-47	
		D_{min}	Short	2_2_2_2_ MR_LG_ DN_SP	139
				3_2_1_2_ MR_LG_ DN_SP	140
				5_B_2_2_ MR_LG_ DN_SP	141
				PG-48	
			Long	4_E_6_2_ MR_LG_ DN_LP	142
				8_B_6_2_ MR_LG_ DN_LP	143
				9_3_6_2_ MR_LG_ DN_LP	144

Chapter 4
Test Program

Tests were conducted to evaluate strength, stiffness, and deformation characteristics of the system under consideration when subjected to simulated seismic loading, and the results were used for validating the numerical models. All testing was performed in accordance with the applicable standards and specifications. According to the FEMA P695 methodology, testing should be conducted at various levels to reliably capture and predict structural response including

- material test data,
- components and connections test data, and
- assembly and system test data.

Material testing was not conducted as part of this study because the data can be obtained from the previous studies. CLT material specifications are available in the ANSI/APA PRG 320 standard, which provides information on performance and requirements for rated CLT. The full test matrix connector testing and wall testing are presented in Sections 4.1 and 4.2.

Both connector and wall tests were conducted using a test protocol specifically developed for light-frame wood fasteners, walls, and other assembly tests developed by Krawinkler and others (2001). Figure 4.1 presents what has become known as the "CUREE test protocol" (CUREE, Consortium of Universities for Research in Earthquake Engineering). The testing protocol was considered for displacement-controlled cyclic testing, and it consists of initiation cycles, primary cycles, and trailing cycles. Initiation cycles at the beginning of the load protocol are to check the data acquisition system and all its components, primary cycles are larger than the preceding cycles, and trailing cycles are 75% of the preceding primary cycle. The reference displacement was obtained from monotonic loading. The reference displacement, Δ, is defined as the deformation where the load drops, for the first time, below 80% of the maximum applied load to the specimen.

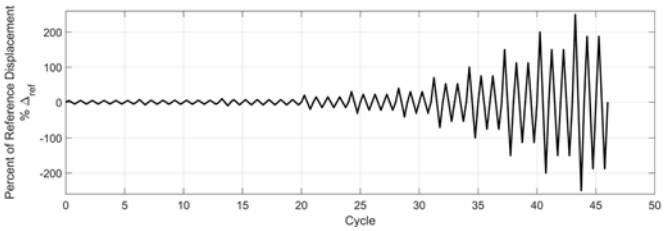


Figure 4.1. CUREE loading protocol (CUREE, Consortium of Universities for Research in Earthquake Engineering).

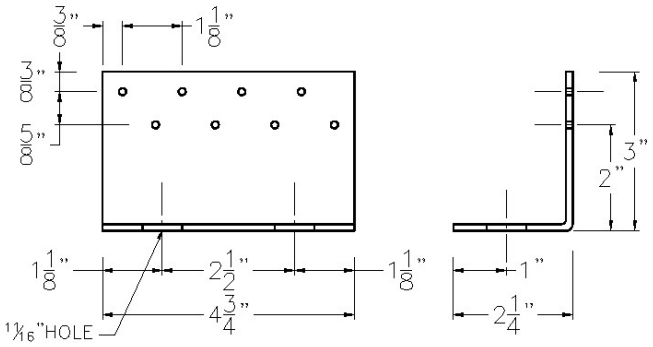


Figure 4.2. A3-type connector.

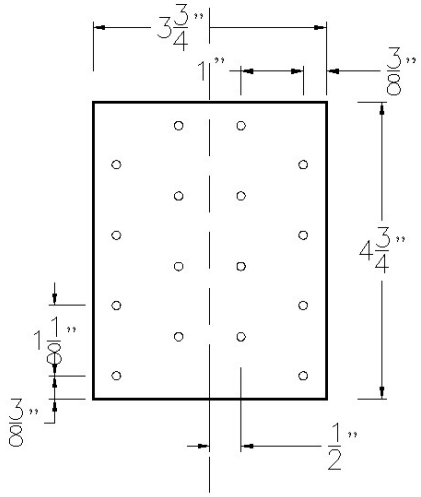


Figure 4.3. Interpanel connector equivalent to straightened A3-type connector.

4.1 Connector Tests

4.1.1 Testing Configuration and Loading Protocol

The connector testing phase is divided into two parts: angle bracket connectors, used for attachment of the wall to the supporting element, and interpanel connectors. Both types of connectors were manufactured from 0.105-in. (2.7-mm) ASTM A653 (ASTM 2017) Grade 33 sheet metal (steel) in the structures laboratory at Colorado State University, Fort Collins, Colorado, USA (CSU) to keep the connector testing as generic as possible. Steel angle brackets and interpanel connectors are shown in Figures 4.2 and 4.3. The A3 connector used eight 16d box nails (3-1/2 by 0.135 in. (89 by 3.4 mm)) with bolts prepared from ASTM 1554 (ASTM 2018a) Grade A36 steel designed per the NDS (AWC 2015a). The metal bracket transfers all the imposed deformation to the nails, which are designed to have Mode III or IV yielding behavior under lateral load and eventually pull out of the CLT panel to ensure predictable nonlinear behavior of the fasteners. The interpanel connector, shown in Figure 4.3, is equivalent to the A3 angle brackets in terms of the number of nails used in the connector. The test

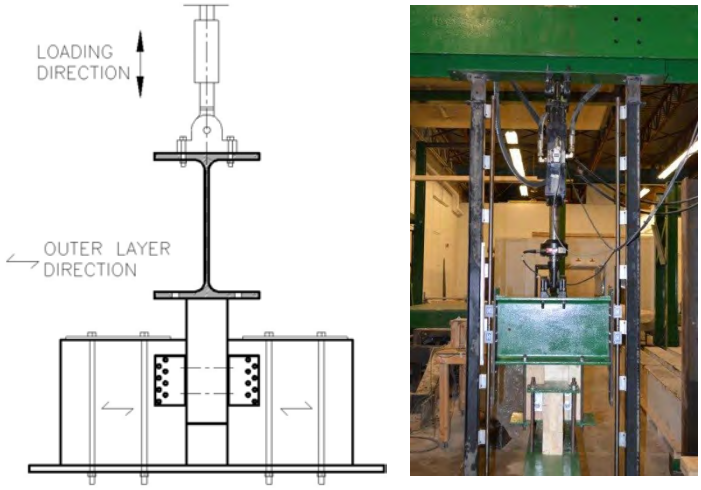


Figure 4.4. Connector shear test.

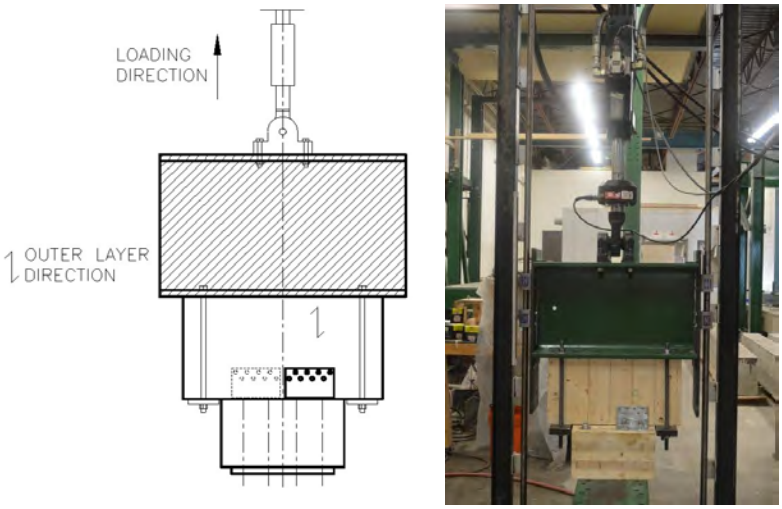


Figure 4.5. Connector uplift test.

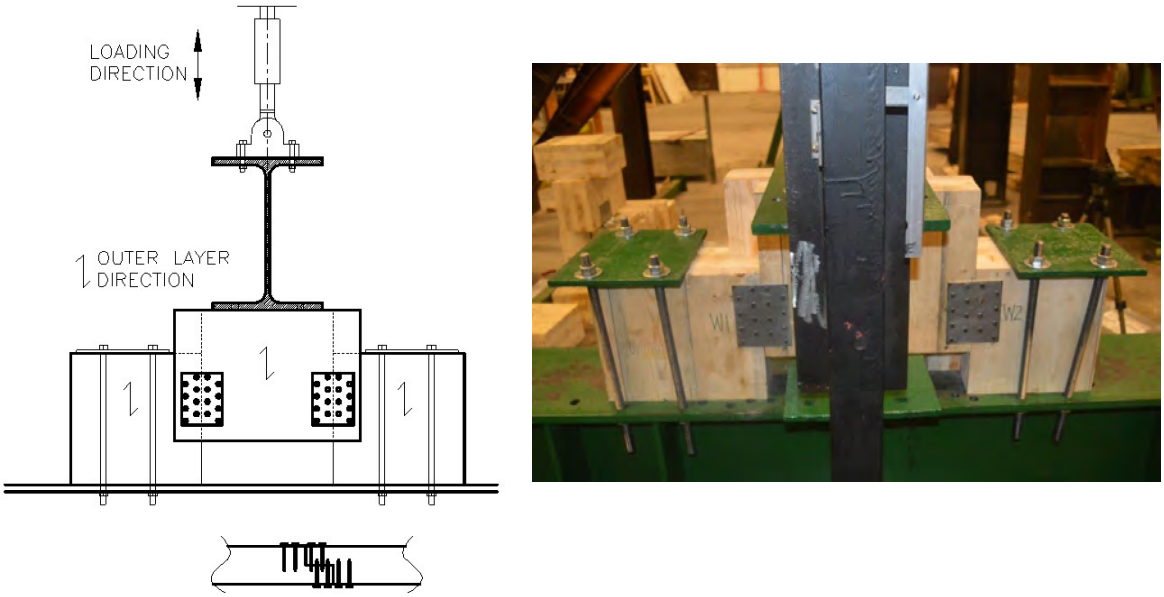


Figure 4.6. Interpanel connector shear test.

Table 4.1—Connector test matrix (1 in. = 25.4 mm)

Test type	Connector type	CLT grade	Tests
Shear	A3- (8)16d box (3-1/2- by 0.135-in.) nails in vertical leg and two 5/8-in.-diameter rods (ASTM 1554 Grade A36 steel) in horizontal leg	E1	One monotonic and 10 cyclic
		V2	
Uplift	A3- (8)16d box (3-1/2- by 0.135-in.) nails in vertical leg and two 5/8-in.-diameter rods (ASTM 1554 Grade A36 steel) in horizontal leg	E1	One monotonic and 10 nonreversed
		V2	
Shear	Interpanel (8)16d box (3-1/2- by 0.135-in.) nails on each side connector center line	E1	One monotonic and 10 cyclic
		V2	

configurations for both types of connectors are shown in Figures 4.4 to 4.6. Steel angle brackets were tested under shear and uplift, and interpanel connectors were tested under shear only. Uplift tests were conducted in a similar manner. However, in this case, specimens were subjected to a single-sided (not reversed-cyclic) CUREE-like protocol because of the in situ boundary conditions.

Two different grades of CLT, E1 and V2, based on ANSI/APA PRG 320 were considered in testing. E indicates that parallel layers are E-rated or MSR laminations, and V indicates that parallel layers are visually graded laminations. To capture statistical variability in the tests, one monotonic and ten cyclic tests were performed for each configuration. The summary of connector tests is provided in Table 4.1.

4.1.2 Connector Testing Results

A3 connectors performed as intended, and nail yielding and withdrawal were observed, as shown in Figure 4.7. The testing showed that the nonlinear behavior was primarily that of the fasteners. Similarly, uplift tests are shown in Figure 4.8. Monotonic and a sample hysteretic plot for A3 connectors are shown in Figures 4.9 to 4.12, and all the shear and uplift test results are summarized in Tables 4.2 and 4.3.

Similar to the shear tests for the A3-type connector, testing was done to determine the capacity of interpanel connectors with two different grades of CLT, E1 and V2. A connector test is shown in Figure 4.13, and hystereses are provided in Figures 4.14 and 4.15 for E1 and V2 grades, respectively. A comparison of interpanel connector and A3-type connector shear tests is provided in Figure 4.16. A summary of the test results is provided in Table 4.4.

Tests results were analyzed based on the procedure in FEMA P795 (FEMA 2011), which is similar to ASTM E2126 (ASTM 2009) except for the definition of Δ_{yield}. FEMA P795 is the complementary methodology to FEMA P695 and will probably be used for connectors to show performance equivalence after the FEMA P695 process is completed. An example of the Test 14 backbone curve is provided in Figure 4.17, and the individual test average parameters for the positive and negative excursions are reported in Table 4.5.

4.2 Isolated Wall Testing

4.2.1 Test Setup and Loading Protocol

CLT isolated wall tests were performed with the same connectors used in the connector testing. The purpose of these tests was to systematically investigate the influence of various parameters on behavior of the wall in terms of strength, stiffness, ductility, and energy dissipation. The parameters were

- boundary conditions,
- presence of gravity loading,
- connector type,
- connector thickness,
- CLT grade,
- CLT panel aspect ratio,
- panel thickness, and
- presence of interpanel connector (vertical joint).

The main design assumptions for these walls, as dictated by the design approach, are that all overturning is resisted by overturning anchors (tie rod or hold-downs) at wall ends and that shear is resisted by the angle brackets. These assumptions were also adopted in the initial stages of the SOFIE project (Ceccotti 2008), and they align well with the already established method for light-frame wood shear walls. A comparison of different analytical models with experimental data (Gavric and others 2015) showed this approach to be conservative.

The general test setup is shown in Figure 4.18. Vertical actuators under load control were used for gravity load, and horizontal actuators under displacement control were used for application of the shear load. Because the vertical actuators were in force control, the change in angle did not change the applied vertical loads. However, the horizontal component of the load applied as a result of the angle change was accounted for during post-processing of the test

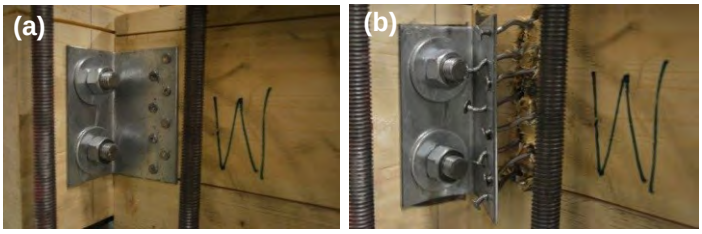


Figure 4.7. A3-type connector (a) before and (b) after test.



Figure 4.8. A3-type connector (a) before and (b) after uplift test.

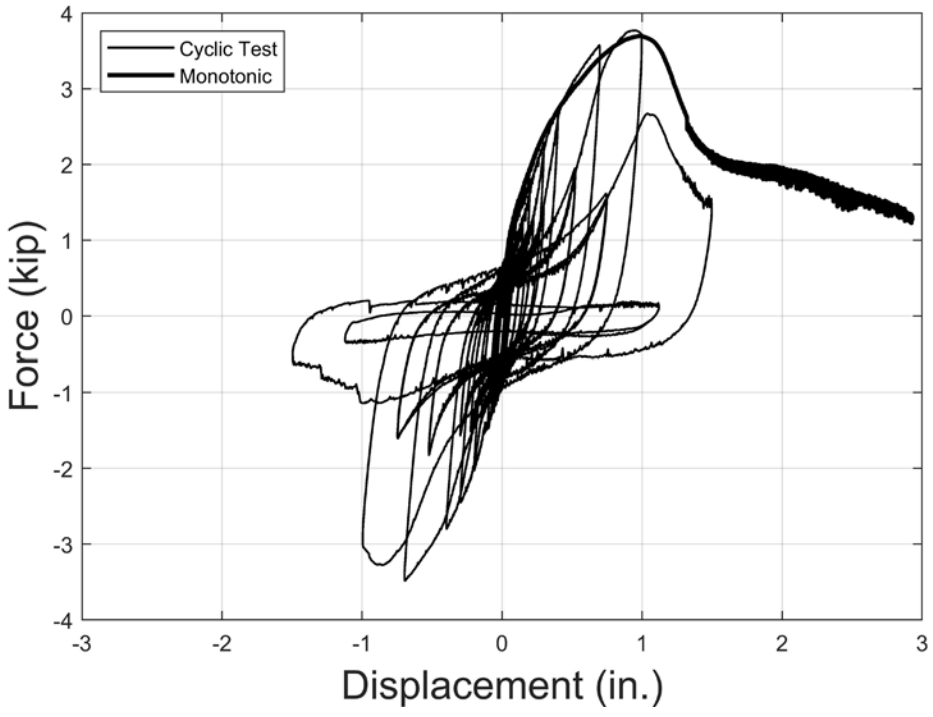


Figure 4.9. A3 connector, shear test, EI grade (1 kip = 1,000 lb= 0.45 metric ton; 1 in.= 25.4 mm).

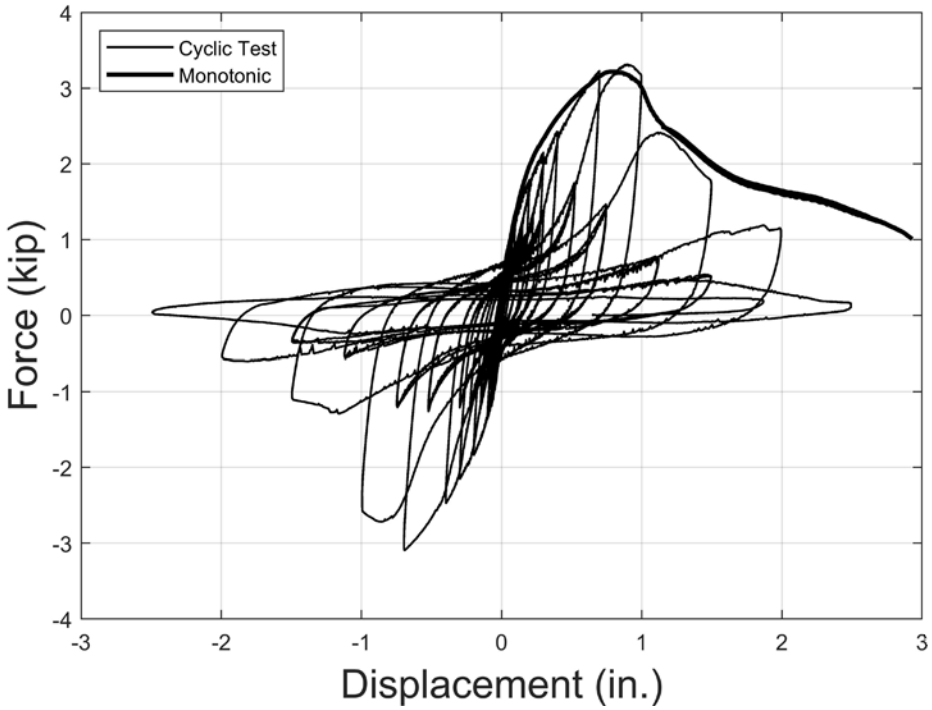


Figure 4.10. A3 connector, shear test, V2 grade (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

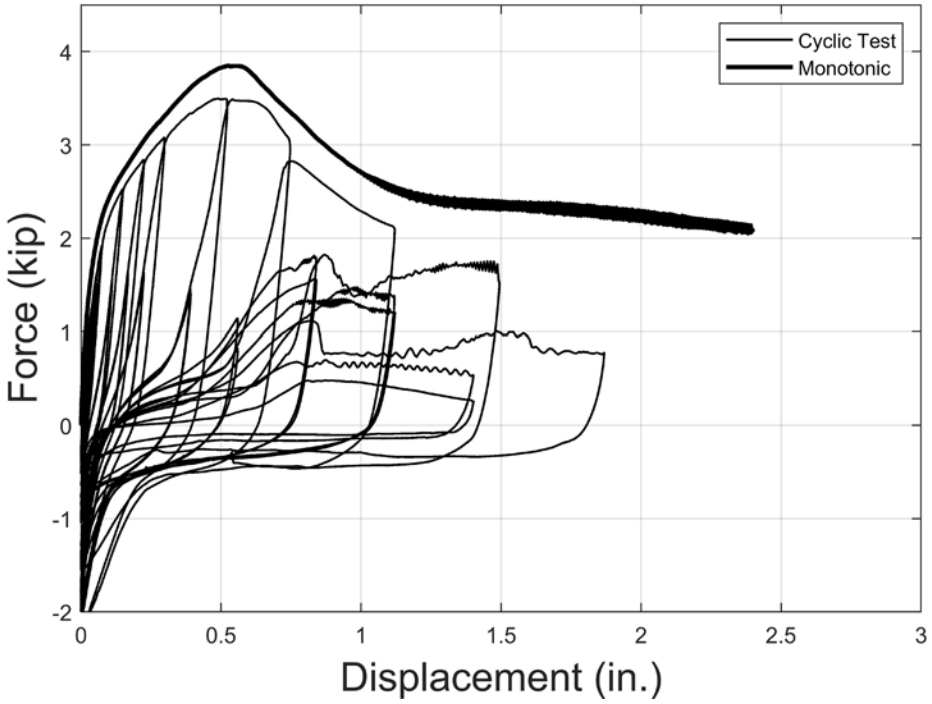


Figure 4.11. A3 connector, uplift test, El grade (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

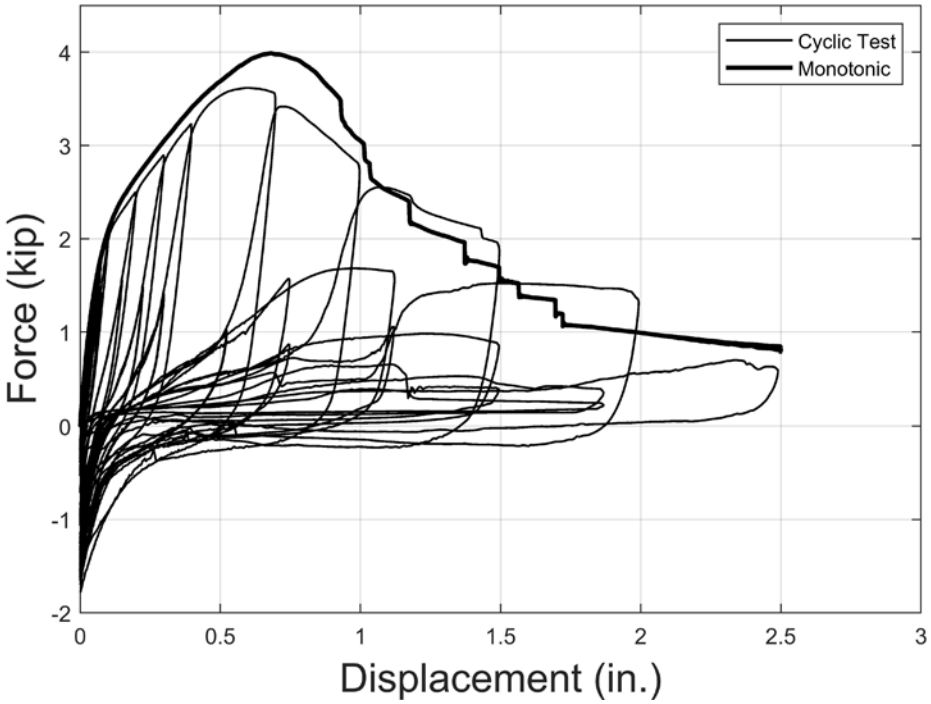


Figure 4.12. A3 connector, uplift test, V2 grade (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Table 4.2—Connector shear test results (1 kip = 1,000 lb = 0.45 metric ton)

V2 grade			E1 grade		
Test No.	Max. load (kip)	Min. load (kip)	Test No.	Max. load (kip)	Min. load (kip)
7	3.31	–3.09	30	3.79	–3.7
8	3.72	–3.06	31	3.84	–3.77
9	3.27	–2.99	32	3.89	–3.9
10	3.34	–3.27	33	3.94	–3.7
11	3.28	–3.01	34	3.69	–3.63
12	3.53	–3.08	35	3.37	–3.33
13	3.40	–3.19	36	4.12	–3.52
14	3.68	–3.30	37	3.76	–3.56
15	3.19	–3.01	38	3.77	–3.48
16	3.35	–2.89	39	4.04	–3.55
Avg.	3.41	–3.09	Avg.	3.82	–3.61
Std. dev.	0.18	0.13	Std. dev.	0.21	0.16
COV	0.053	0.042	COV	0.055	0.044
95% CI	±0.13[a]	±0.09	95% CI	±0.15	±0.12

[a]This is calculated based on $t_{\alpha/2}s/\sqrt{n}$; the upper and lower limit is thus 3.41±0.13.

Table 4.3—Connector uplift test results (1 kip = 1,000 lb = 0.45 metric ton)

V2 grade		E1 grade	
Test No.	Max. load (kip)	Test No.	Max. load (kip)
72	3.53	61	3.86
73	3.97	62	3.63
74	4.24	63	3.57
75	3.83	64	3.76
76	3.76	65	3.60
77	3.64	66	3.50
78	3.69	67	3.60
79	4.07	68	3.65
80	3.62	133	4.38
81	3.61	134	4.64
Avg.	3.80	Avg.	3.82
Std. dev.	0.23	Std. dev.	0.38
COV	0.061	COV	0.099
95% CI	±0.17	95% CI	±0.27

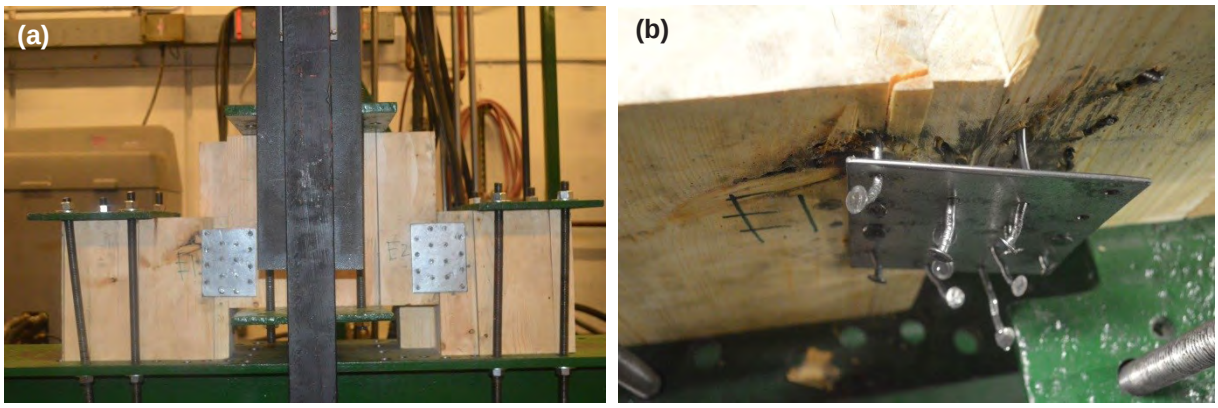


Figure 4.13. Interpanel connector (a) before and (b) after test.

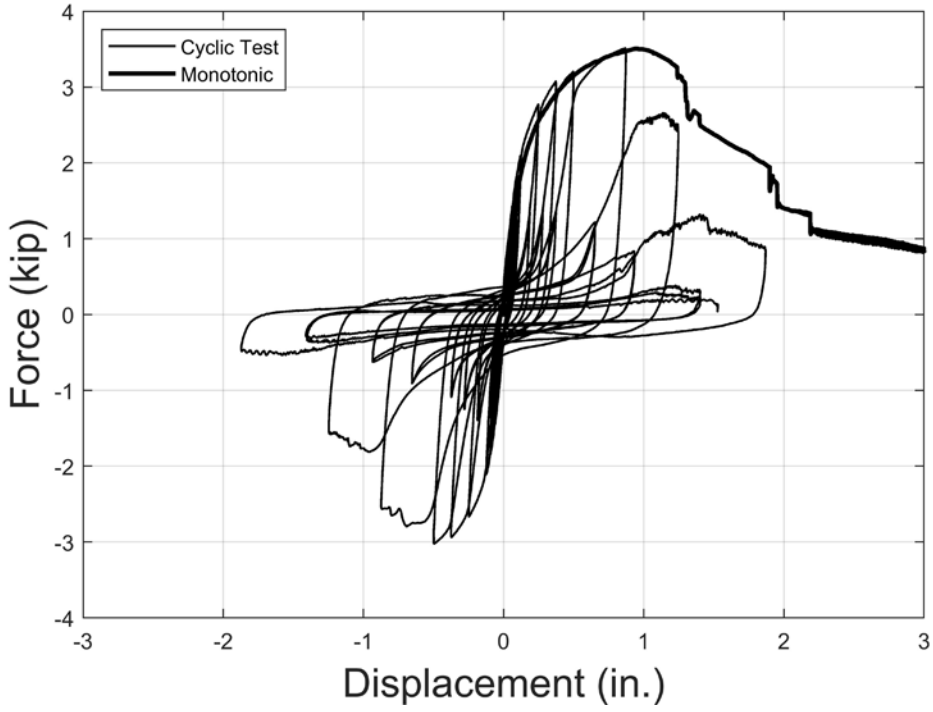


Figure 4.14. E3 connector, shear test, El grade (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

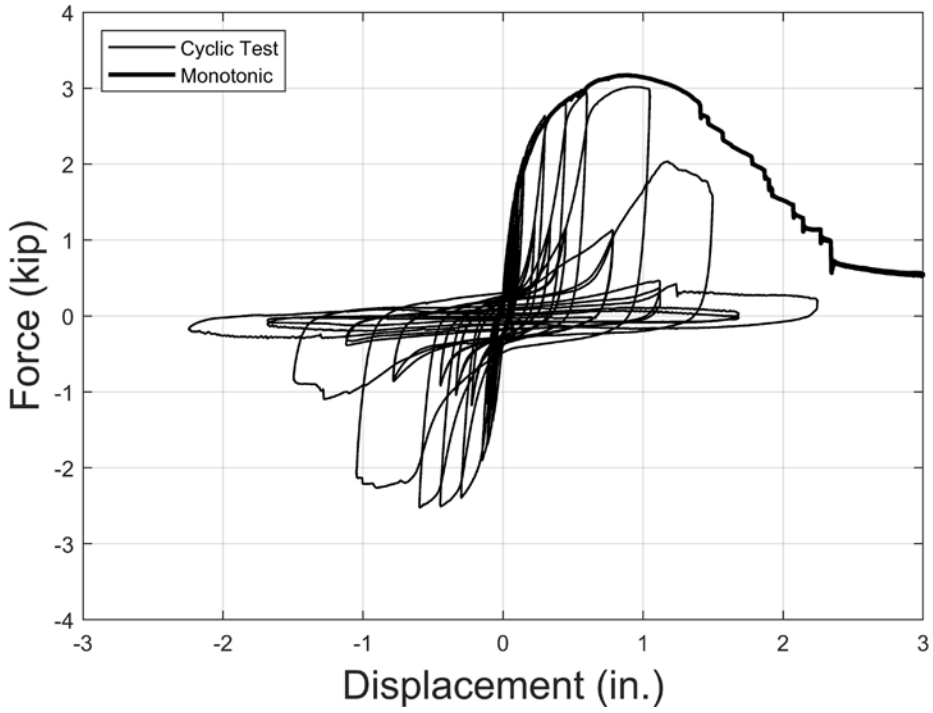


Figure 4.15. E3 connector, shear test, V2 grade (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

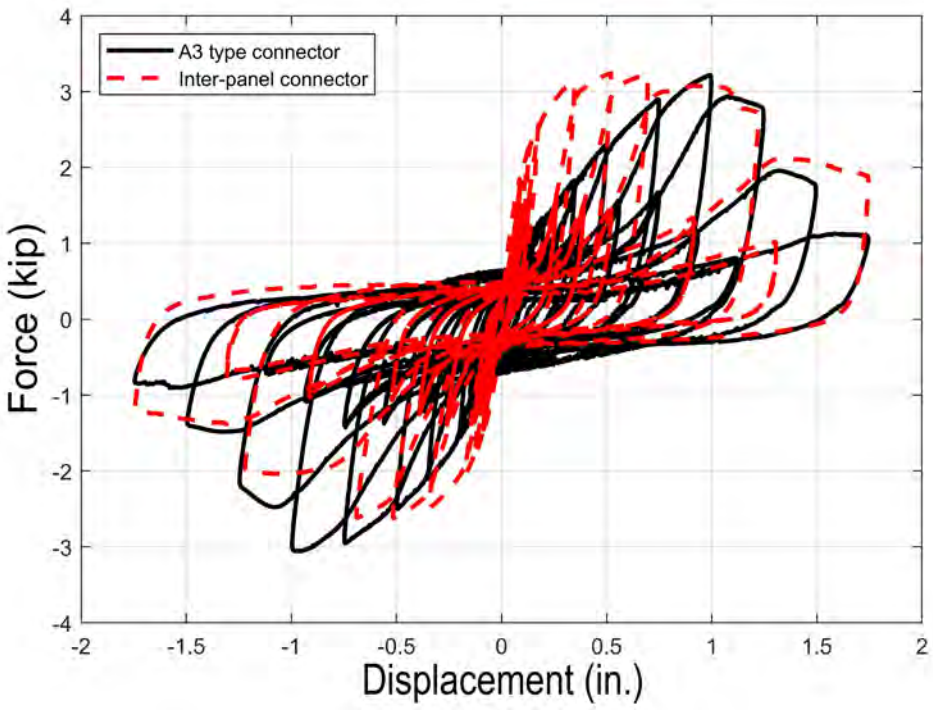


Figure 4.16. Interpanel connector vs. A3-type connector (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Table 4.4—Shear test results for E3-type interpanel connector (1 kip = 1,000 lb = 0.45 metric ton)

V2 grade			E1 grade		
Test No.	Max. load (kip)	Min. load (kip)	Test No.	Max. load (kip)	Min. load (kip)
97	3.15	–2.56	84	3.64	–3.35
98	3.02	–2.52	85	3.59	–3.03
99	3.03	–2.57	86	3.49	–2.94
100	3.11	–2.79	88	4.09	–3.29
101	3.38	–2.96	90	3.42	–2.93
102	3.25	–2.84	91	3.88	–3.40
103	3.50	–2.91	92	4.14	–3.15
104	2.79	–2.66	93	3.75	–2.92
105	3.10	–2.80	94	3.51	–3.02
106	3.11	–2.64	95	3.54	–2.95
Avg.	3.14	–2.73	Avg.	3.71	–3.10
Std. dev.	0.20	0.15	Std. dev.	0.25	0.19
COV	0.064	0.055	COV	0.067	0.061
95% CI	±0.14	±0.11	95% CI	±0.18	±0.13

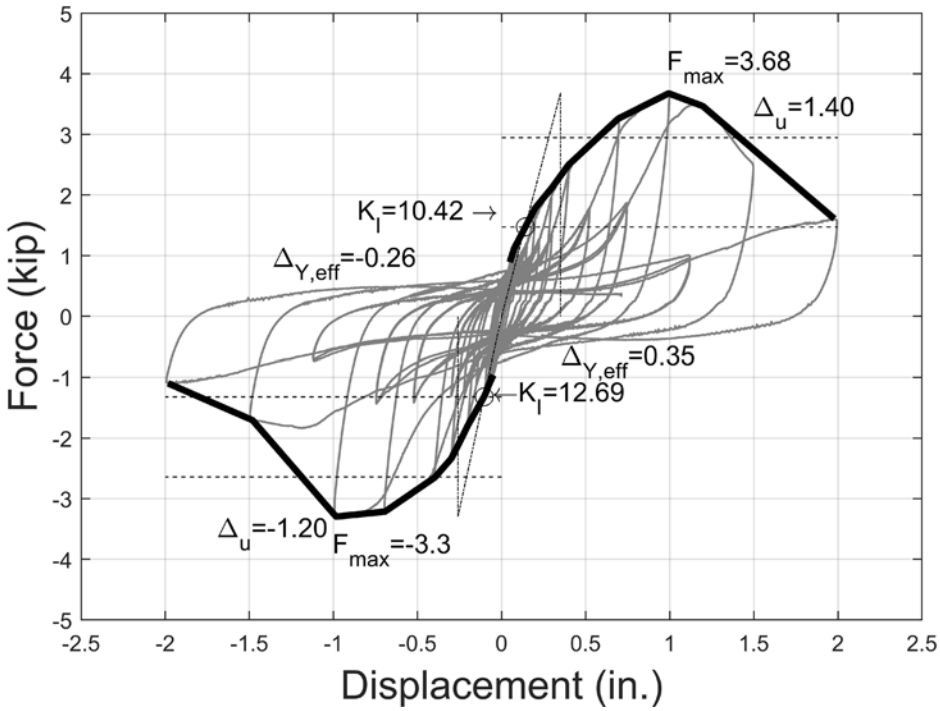


Figure 4.17. Test 14 backbone with parameters calculated based on FEMA P795 (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Table 4.5—Cyclic envelope parameters for connector tests (kip = 1,000 lb = 0.45 metric tons; 1 in. = 25.4 mm)

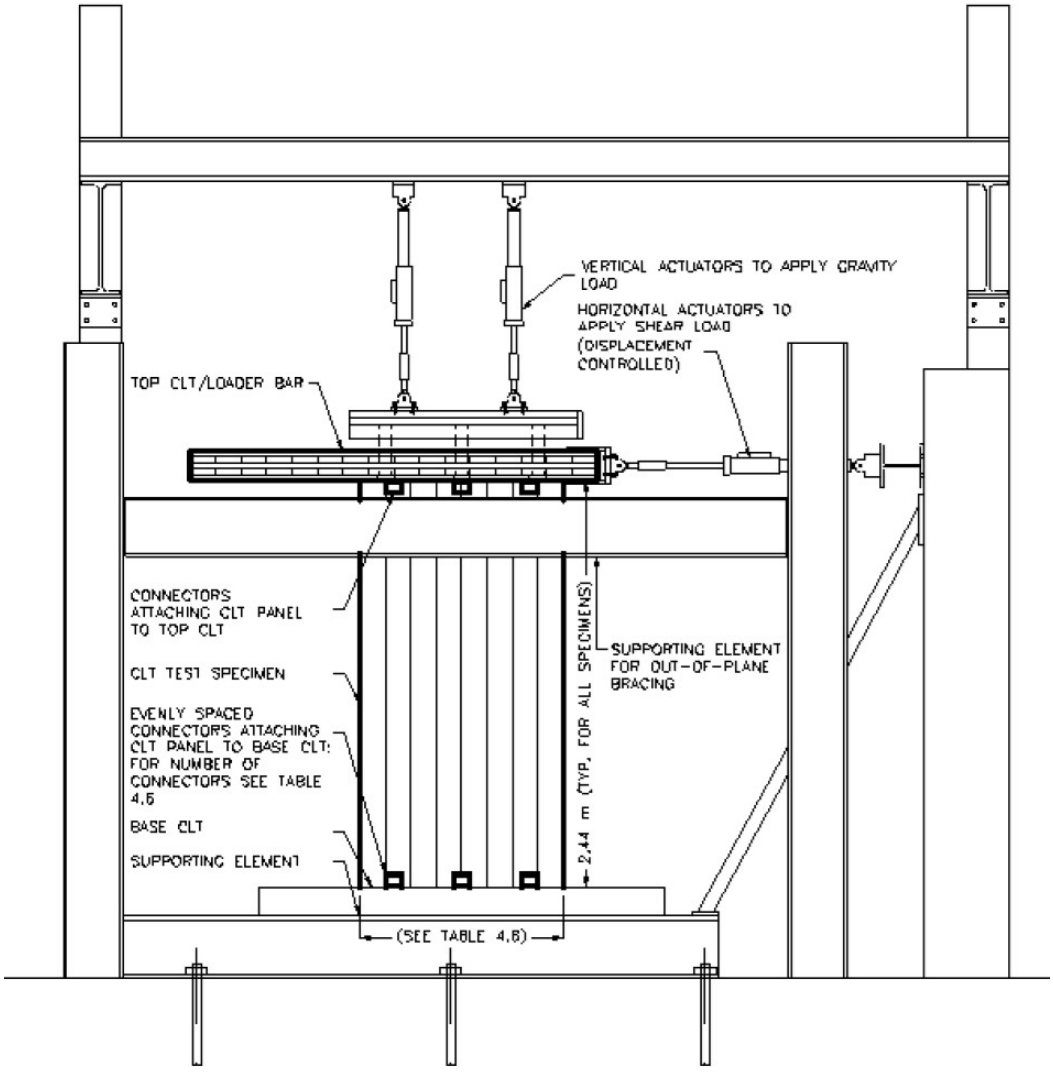
Test type	Test No.	Initial stiffness K_1 (kip/in)	Effective yield $\Delta_{Y,eff}$ (in.)	Ultimate load F_{max} (kip)	Displacement corresponding to ultimate load $\Delta_{F_{max}}$ (in.)	Ultimate deformation (0.8 F_{max}) Δ_u (in.)	Effective ductility capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Shear test A3-type connector, V2 grade	7	13.93	0.23	3.20	0.80	1.01	4.37
	8	15.11	0.23	3.39	0.84	1.07	4.73
	9	13.19	0.24	3.13	0.80	0.99	4.21
	10	14.25	0.24	3.31	0.80	1.04	4.31
	11	14.82	0.22	3.14	0.69	1.10	5.12
	12	14.47	0.24	3.31	0.70	0.99	4.21
	13	10.52	0.31	3.29	0.85	1.06	3.43
	14	11.56	0.31	3.49	0.99	1.30	4.26
	15	10.80	0.30	3.09	0.96	1.26	4.18
	16	7.50	0.44	3.12	0.84	1.10	2.52
	Avg.	12.61	0.27	3.25	0.82	1.09	4.13
	Std. dev.	2.45	0.07	0.13	0.10	0.11	0.71
	COV	0.194	0.259	0.040	0.122	0.101	0.172
	95% CI	±1.76	±0.05	±0.09	±0.07	±0.08	±0.51
Shear test A3-type connector, E1 grade	30	19.01	0.20	3.74	0.69	1.02	5.08
	31	16.95	0.23	3.80	0.84	1.07	4.76
	32	19.88	0.20	3.89	0.84	1.04	5.20
	33	14.45	0.27	3.82	0.84	1.02	3.76
	34	16.33	0.24	3.66	0.83	1.04	4.40
	35	15.33	0.22	3.35	0.69	0.92	4.18
	36	12.72	0.31	3.82	0.84	1.07	3.51
	37	16.26	0.23	3.66	0.93	1.09	4.84
	38	13.67	0.28	3.62	0.83	1.06	3.84
	39	13.72	0.29	3.79	0.84	1.05	3.68
	Avg.	15.83	0.24	3.71	0.82	1.04	4.32
	Std. dev.	2.34	0.04	0.15	0.07	0.05	0.62
	COV	0.148	0.167	0.040	0.085	0.048	0.144
	95% CI	±1.67	±0.03	±0.11	±0.05	±0.03	±0.44

Table 4.5—Cyclic envelope parameters for connector tests (kip = 1,000 lb = 0.45 metric tons; 1 in. = 25.4 mm) (con.)

Test type	Test No.	Initial stiffness K_1 (kip/in)	Effective yield $\Delta_{Y,eff}$ (in.)	Ultimate load F_{max} (kip)	Displacement corresponding to ultimate load $\Delta_{F_{max}}$ (in.)	Ultimate deformation (0.8 F_{max}) Δ_u (in.)	Effective ductility capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Uplift test A3-type connector, V2 grade	72	28.24	0.13	3.53	0.65	0.96	7.38
	73	30.54	0.13	3.97	0.62	0.84	6.46
	74	27.36	0.15	4.24	0.80	1.01	6.73
	75	30.04	0.13	3.83	0.57	0.91	7.00
	76	26.82	0.14	3.76	0.59	0.87	6.21
	77	23.42	0.15	3.63	0.65	0.91	6.07
	78	24.28	0.15	3.69	0.68	1.00	6.67
	79	27.10	0.15	4.07	0.67	1.20	8.00
	80	27.69	0.13	3.60	0.65	0.96	7.38
	81	23.75	0.15	3.61	0.69	0.98	6.53
	Avg.	26.92	0.14	3.79	0.66	0.96	6.84
	Std. dev.	2.47	0.01	0.23	0.06	0.10	0.60
	COV	0.092	0.071	0.061	0.091	0.104	0.088
	95% CI	±1.76	±0.01	±0.17	±0.05	±0.07	±0.43
Uplift test A3-type connector, E1 grade	61	37.56	0.10	3.85	0.50	0.92	9.20
	62	32.09	0.11	3.61	0.63	0.92	8.36
	63	27.08	0.13	3.56	0.63	0.85	6.54
	64	29.96	0.13	3.75	0.62	0.87	6.69
	65	32.12	0.11	3.60	0.57	0.95	8.64
	66	31.37	0.11	3.49	0.44	0.77	7.00
	67	35.95	0.10	3.60	0.51	0.70	7.00
	68	34.57	0.11	3.63	0.54	0.92	8.36
	133	43.75	0.10	4.38	0.67	0.90	9.00
	134	33.34	0.14	4.63	0.72	0.90	6.43
	Avg.	33.78	0.11	3.81	0.58	0.87	7.72
	Std. dev.	4.60	0.01	0.38	0.09	0.08	1.09
	COV	0.136	0.091	0.100	0.155	0.092	0.141
	95% CI	±3.29	±0.01	±0.27	±0.06	±0.06	±0.78

Table 4.5—Cyclic envelope parameters for connector tests (kip = 1,000 lb = 0.45 metric tons; 1 in. = 25.4 mm) (con.)

Test type	Test No.	Initial stiffness K_1 (kip/in)	Effective yield $\Delta_{Y,eff}$ (in.)	Ultimate load F_{max} (kip)	Displacement corresponding to ultimate load $\Delta_{F_{max}}$ (in.)	Ultimate deformation (0.8 F_{max}) Δ_u (in.)	Effective ductility capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Shear test A3-type connector, V2 grade	97	22.71	0.13	2.84	0.50	1.18	9.40
	98	23.75	0.12	2.76	0.82	1.14	9.46
	99	27.82	0.10	2.79	0.80	1.08	10.80
	100	27.34	0.11	2.95	0.58	0.94	8.50
	101	35.24	0.09	3.17	0.52	1.02	11.33
	102	29.70	0.11	3.05	0.76	0.91	8.67
	103	23.91	0.14	3.21	0.61	0.93	6.85
	104	27.61	0.10	2.72	0.46	0.90	9.00
	105	24.47	0.12	2.93	0.83	1.14	9.46
	106	23.82	0.13	2.88	0.45	0.99	7.92
	Avg.	26.64	0.11	2.93	0.63	1.02	9.14
	Std. dev.	3.80	0.01	0.17	0.15	0.10	1.30
	COV	0.143	0.091	0.058	0.238	0.098	0.142
	95% CI	±2.72	±0.01	±0.12	±0.11	±0.07	±0.93
Shear test A3-type connector, E1 grade	84	29.67	0.12	3.49	0.67	1.27	11.00
	85	21.37	0.16	3.31	0.87	0.97	6.26
	86	23.77	0.14	3.22	0.87	0.99	7.30
	88	33.46	0.11	3.69	0.66	1.09	9.86
	90	21.68	0.15	3.18	0.86	1.12	7.69
	91	24.86	0.15	3.64	0.87	1.25	8.33
	92	25.88	0.14	3.65	0.83	1.01	7.18
	93	31.10	0.11	3.32	0.88	1.12	10.67
	94	25.43	0.13	3.27	0.68	0.99	7.62
	95	25.05	0.13	3.24	0.85	1.01	8.04
	Avg.	26.23	0.13	3.40	0.80	1.08	8.39
	Std. dev.	3.98	0.02	0.20	0.09	0.11	1.58
	COV	0.152	0.154	0.059	0.113	0.102	0.188
	95% CI	±2.84	±0.01	±0.14	±0.07	±0.08	±1.13



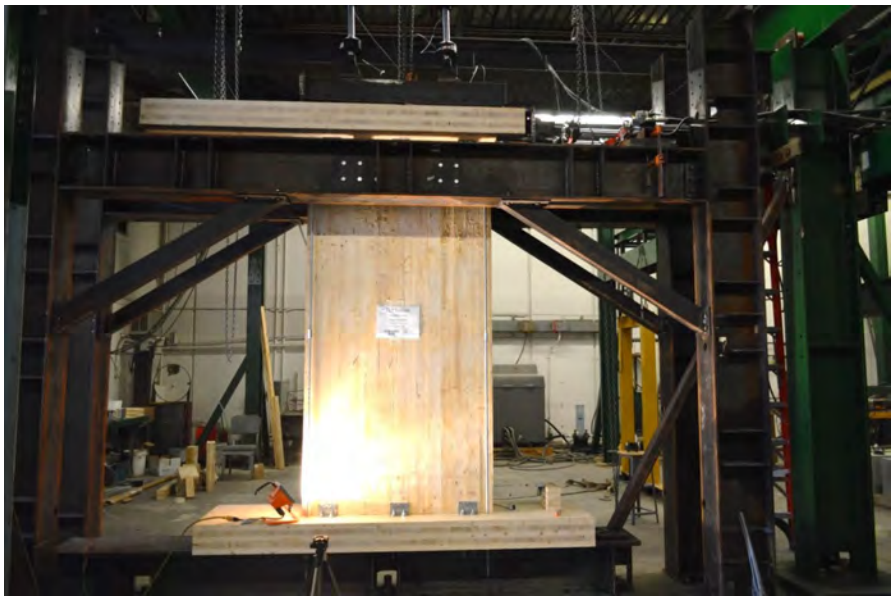


Figure 4.18. Isolated wall test setup.

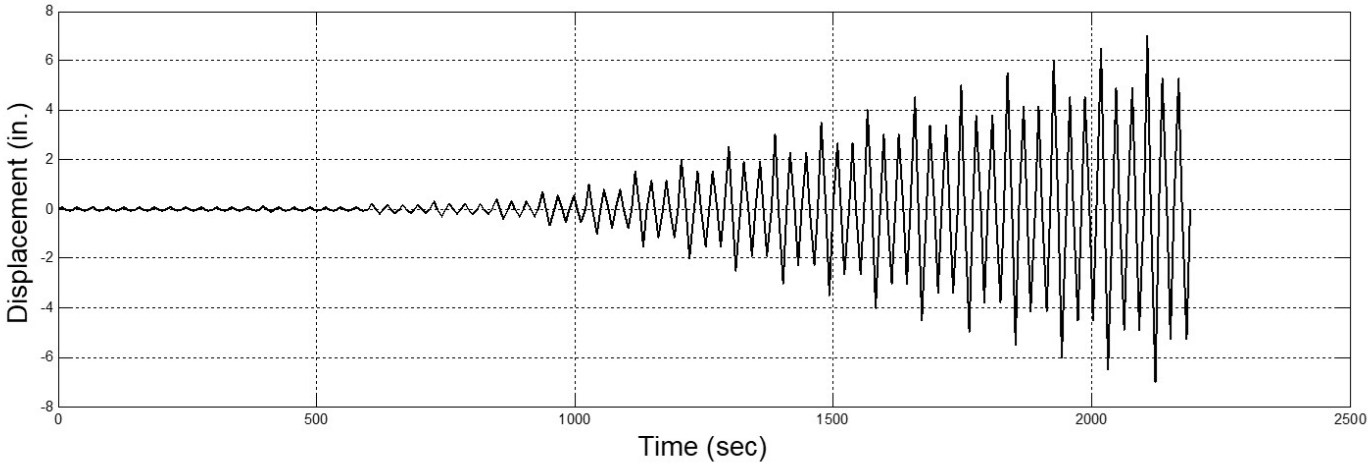


Figure 4.19. Isolated wall test loading protocol (1 in. = 25.4 mm).

results, and the restoring force of the walls was adjusted on the hysteretic plot. The top CLT panel was used as the loader bar to simulate the floor diaphragm, and the bottom CLT was used as the base. Lateral braces were provided (not shown in Figure 4.18 for clarity) to prevent out-of-plane movement of the panels. The CUREE loading protocol was used for all the reverse cyclic loading with reference displacement, Δ_{ref}, of 1 in. and is shown in Figure 4.19. Loading was applied at a rate of 0.033 Hz (30-s cycles). Although the cyclic frequency rate used was slower than recommended in ASTM E2126 to avoid inertial effects, the loading rate for the loading protocol fell within the recommended values of 0.04 to 2.5 in/s. This slower cyclic frequency rate meets the standard and also avoids inertial effects. Table 4.6 provides information on CLT wall tests and includes CLT grade, geometry, vertical load, and the applied boundary condition.

4.2.2 Test Configurations and Results

Summarized load data are provided in Table 4.7. CLT wall test results were also analyzed based on the procedure in FEMA P795, which is similar to ASTM E2126 except for the definition of Δ_{yield}. The average parameters for the positive and negative excursions are reported in Table 4.8.

Table 4.9 presents general information and purpose of the attached instrumentation. Placement of these instrumentations for single-panel and multipanel configuration is shown in Figures 4.20 and 4.21.

4.2.3 Test Results Discussion

4.2.3.1 Failure Mechanism

All testing configurations with A3 connectors performed well under cyclic loading. Failure mechanisms observed in Tests 01 and 02 conducted with A1-type angle brackets were not ideal, which is discussed in detail in the next section.

The damage for all the configurations was mainly concentrated in the base connectors attaching the wall to the base CLT used to transfer the shear load. Fastener yielding and withdrawal were observed with no signs of fatigue, and interpanel connectors (connectors along the vertical joint) also showed nail yielding and withdrawal. Photos of Test 05 are shown in Figure 4.22 for the purpose of discussion. As shown in Figure 4.22, the damage was concentrated in the base connectors, and nail yielding and withdrawal were observed. One can see the effect of panel rocking on the fasteners by comparing the middle connector to the two edge connectors. Nail withdrawal was significantly greater in the edge connectors. Also observed was CLT wood crushed perpendicular to the grain at the bottom toe of the wall and the top corner as a result of rocking.

For the 2:1 aspect ratio panel, the damage was caused by a combination of sliding and rocking, whereas in the case of the 4:1 aspect ratio panels, the damage was mainly caused by rocking. The effect of sliding on hysteresis is evident by looking at hysteresis provided for Tests 03 and 14 in Figures C3 and C11, respectively (in Appendix C). The corresponding edge connectors before failure are shown in Figures 4.23 and 4.24. It is important to note that both tests were stopped when complete nail withdrawal was observed in the base connectors and, although both were 2:1 aspect ratio panels, Test 03 experienced more sliding. The fact that nail withdrawal caused by sliding was sudden as opposed to the gradual withdrawal caused by rocking led to the rapid loss in strength observed in the hysteresis.

As mentioned earlier, in the case of the 4:1 aspect ratio panels (Tests 21, 23, and 26), the lateral displacement was primarily caused by rocking. This is shown in Figures 4.25 to 4.27.

For multipanel configurations (Tests 23 and 26), the rocking behavior resulted in withdrawal followed by shear failure of some nails in the interpanel connectors, which is shown in Figure 4.28.

4.2.3.2 Connector Thickness

Tests 01 and 02 were conducted using the A1-type angle bracket with dimensions similar to the A3-type connector

Table 4.6—CLT wall matrix (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m)

Test No.	Grade	Height (ft)	Length (ft)	No. of plys	Thickness (in.)	No. of connectors	Connector type	Gravity load (kip/ft)	Hold-down rod (number of and size in in.)
1[a,b,c]	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
2[a,b,c]	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
3	V2	8	4	5	6.65	3	A3	0.68	(2) 5/8
4	V2	8	4	5	6.65	3	A3	1.28	(2) 5/8
5	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
6[a]	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
9	V2	8	4	5	6.65	3	A3	—	(2) 5/8
10	V2	8	4	3	3.9	4	A3	—	(2) 5/8
11	V2	8	4	5	6.65	2	A3	—	(1) 5/8
13	E1	8	4	5	6.89	2	A3	—	(1) 5/8
14	E1	8	4	5	6.89	3	A3	—	(2) 5/8
15	E1	8	4	5	6.89	2	A3	—	(1) 5/8
17	E1	8	4	5	6.89	4	A3	—	(2) 5/8
18	V2	8	4	3	3.9	2	A3	—	(1) 5/8
19	V2	8	4	3	3.9	5	A3	—	(2) 3/4
20	V2	8	4	7	9.41	5	A3	—	(2) 3/4
21	V2	8	2	3	3.9	2	A3	—	(2) 5/8
22[d]	V2	8	8	3	3.9	4	A3	—	(1) 5/8
23	V2	8	2 (2)	5	6.65	4	A3	—	(2) 5/8
26	V2	8	4 (2)	5	6.65	8	A3	—	(2) 5/8

[a]Test 1, 2, and 6 were performed with the imposed boundary condition. The imposed boundary condition is explained in detail in Section 4.2.3.3.
[b]Test 1 and 2 were performed during the exploratory phase of the A-type connector thickness.
[c]The top and bottom CLT panels matched the CLT wall panel grade in all testing, except for Test 1 and 2 where the wall panels were of V2 grade and the top and bottom CLT panels were of E1 grade. Top and bottom CLT panels of E1 and V2 grades were 6.89 and 6.65 in. thick, respectively.
[d]Test 22 was (8- by 8-ft) 1:1 aspect ratio wall that was not covered by the design methodology.

and a thickness of 3/8 in. (9.53 mm). The connector thickness led to nail shear failure as opposed to combined nail yielding and withdrawal, which was the intended mode of failure as a result of connector thickness. Although nail yielding occurred as well as wood crushing around the nail, the lack of any noticeable withdrawal was considered undesirable under fully reversed cyclic loading conditions. This type of shear failure is shown in Figure 4.29. Therefore, the remaining tests were conducted using A3-type connectors. Also, comparing the results obtained from Tests 01 and 02 with the results from other tests, one can see that the deformation capacity is greatly affected by the nail shear failure mechanism. This type of failure coupled with no yielding of the metal connector is associated with reduced shear wall deformation capacity.

4.2.3.3 Boundary Conditions

Based on FEMA P695, the test boundary conditions should be representative of typical construction provided it does not provide any beneficial effects (i.e., that would not be guaranteed to be present in situ). In the case of CLT walls, an important boundary condition is the interface between the wall and the floor or ceiling diaphragm. The stiffness of the diaphragm is believed to affect the wall behavior under cyclic loading because the diaphragm in a structure may be larger than the walls and therefore may remain relatively horizontal throughout the loading. This in turn creates a gap between the wall panel and the diaphragm during the lateral loading and affects the rocking of the CLT wall. To quantify the effect of a top boundary condition, modifications were made to the original test setup, shown in Figure 4.18, to include the effect of a top diaphragm into the wall test. This was done by adding supports to allow sliding of the top CLT panel while keeping it horizontal during the shear loading. The supports, shown in Figure 4.30, consisted of four load cells on each end with acetal polymer plates on top. Load cells were added to determine the effect of friction and to consider friction in post-processing the results and developing the corresponding hysteresis.

To determine the coefficient of friction of the acetal polymer plates, a total of 10 tests, each with three levels of increasing vertical load, were performed to estimate

Table 4.7—CLT shear wall testing summarized load data results (1 ft = 0.3 m; 1 kip = 1,000 lb = 0.45 metric ton)

Test No.	Length (ft)	No. of connectors	Connector type	F_{max} (kip)	
				positive	negative
1	4	3	A1	12.29	10.15
2	4	3	A1	11.89	10.73
3	4	3	A3	14.79	14.62
4	4	3	A3	15.7	14.22
5	4	3	A3	17.9	18.15
6	4	3	A3	19.51	18.02
9	4	3	A3	15.11	11.62
10	4	4	A3	15.2	13.12
11	4	2	A3	8.03	8.7
13	4	2	A3	7.597	10.72
14	4	3	A3	19.98	16.52
15	4	2	A3	10.16	11.02
17	4	4	A3	24.5	21.9
18	4	2	A3	6.74	7.3
19	4	5	A3	17.93	16.43
20	4	5	A3	19.11	18.69
21	2	2	A3	7.19	5.95
23	2 (2)[a]	4	A3	13.34	13.84
26	4 (2)[b]	8	A3	23.72	24.3

[a]Multipanel configuration, two 2-ft panels.
[b]Multipanel configuration, four 2-ft panels.

a friction coefficient. Friction test results are given in Table 4.10. After the results were obtained for the friction tests, two specific tests, 05 and 06, were conducted to investigate the effect of adding this boundary condition on the CLT wall behavior (i.e., the resulting hysteresis). Test 05 was performed without the imposed boundary condition, whereas Test 06 included the boundary condition, and thus the force values obtained from Test 06 were adjusted for friction based on the friction coefficient. Connector failure in the wall test is shown in Figure 4.31, and the hysteresis for both of these tests is provided in Figure 4.32. From inspection of the hysteresis plots, it was found that the test without the boundary condition imposed produced load deformation responses similar to the test with the boundary condition, with only slight differences in strength, stiffness, and displacement capacity. As a result, additional testing was conducted using the less complex test setup without the boundary condition imposed.

4.2.3.4 Gravity Load

Gravity load can also affect the CLT wall component behavior, and therefore, a number of tests were performed to determine its effect on the isolated CLT wall tests. A 4- by 8-ft (1.22- by 2.44-m) CLT wall under three levels of vertical load (no gravity, 0.68 kip/ft (9.9 kN/m), and 1.28 kip/ft (18.7 kN/m)) was tested, and the results for all three tests are shown in Figure 4.33. These tests were Tests 09, 03, and 04, respectively, in Table 4.6. From Figure 4.33, one can see that an increase in gravity leads to an increase in stiffness of the panel and a slight increase in strength. As a result, gravity load was removed from the remainder of the tests to be conservative.

4.2.3.5 CLT Grade

The effect of CLT grade was investigated by comparing the results of Test 09 with Test 14 and results from Test 10 with Test 17, although the thicknesses are different in the case of the latter comparison. Results are shown in Figures 4.34 and 4.35. Based on the hysteresis, as one would expect, CLT grade had an influence on strength and stiffness of the CLT panels when the exact same connectors and fasteners were used. A similar trend was observed by comparing Tests 11 and 15, shown in Figure 4.36. The strength of wood connections is significantly influenced by the specific gravity of the wood; the higher the specific gravity, the higher the strength and stiffness values for the fasteners. The effect of CLT grade was partly attributed to the specific gravity of these different grades of CLT, which were determined in accordance with ASTM D2395 (ASTM 2014b). E1 grade was found to have on average a higher specific gravity than the V2 grade. Specified specific gravity for each grade is 0.42 in accordance with NDS;

however, measured values were 0.50 and 0.49 for E1 grade and 0.45 and 0.44 for V2 grade for outer and inner layers, respectively.

4.2.3.6 CLT Panel Thickness

Tests 19 and 20 were performed to examine the effect of panel thickness on overall wall behavior. Because CLT is a rocking system, the effect of compression perpendicular to the grain is thought to have an effect on the rocking behavior. Figure 4.37 indicates that there is only a slight difference in the initial stiffness and maximum strength of two panels with different thicknesses with the thicker panel being the stronger and stiffer of the two. A similar trend was observed by comparing the results of Tests 11 and 18, shown in Figure 4.38; albeit in this case, the difference was less significant.

4.2.3.7 Panel Aspect Ratio

To determine the effect of panel aspect ratio, Tests 18 and 21 were performed and the hysteresis was compared in Figure 4.39. Results indicated that although the higher aspect ratio panel (4:1) exhibited less stiffness and somewhat less strength than the lower aspect ratio panel (2:1), it had more deformation capacity and pinched significantly. This added deformation capacity can be attributed to the rocking behavior of the panel as opposed to the rocking and sliding mechanism of other panels tested.

4.2.3.8 Interpanel Connectors

The influence of interpanel connectors was examined by comparing Tests 23 and 10. Although panel thicknesses were different in these two tests, it was previously shown that this was not significant. As seen in Figure 4.40, interpanel connectors added to the deformation capacity of the wall comprised of higher aspect ratio panels, but the two walls (with and without interpanel connectors) remained very close to each other in peak capacity with only a slight reduction in the wall with interpanel connectors. This was probably caused by reduced stiffness of the interpanel connectors relative to the hold-downs, which led to more uplift demand on the base connectors as the panel rocked. Connectors for the vertical joints provided equivalent shear capacity to that of the angle brackets used in the base and top of the wall, which were A3-type connectors in this case. The vertical joint was designed to yield before the shear capacity of the base connectors was reached, resulting in rocking of the individual panels, and this rocking behavior was intended as part of the CLT shear design method. This behavior was observed in Tests 23 and 26 and is shown in Figures 4.41 and 4.42, respectively. The hysteresis for Test 26 is shown in Figure 4.43, which shows that vertical joints added to the deformation capacity of the wall.

Detailed descriptions of the tests with corresponding hysteresis are presented in Appendix C.

Table 4.8—CLT shear wall testing cyclic envelope parameters (1 kip/in = 175 kN/m; 1 in. = 25.4 mm; 1 kip = 1,000 lb = 0.45 metric ton)

Test No.	Initial stiffness K_1 (kip/in)	Effective yield $\Delta_{Y,eff}$ (in.)	Ultimate load F_{max} (kip)	Displacement corresponding to ultimate load $\Delta_{F_{max}}$ (in.)	Ultimate deformation (0.8 F_{max}) Δ_u (in.)[a]	Effective ductility capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
1	7.96	1.46	11.30	2.44	2.50	1.71
2	6.41	1.79	11.42	2.47	2.60	1.46
3	6.70	2.20	14.71	3.72	4.13	1.88
4	6.45	2.32	14.96	3.71	4.03	1.73
5	8.52	2.13	18.04	4.83	6.00	2.82
6	6.58	3.03	19.12	4.21	5.88	1.94
9	4.51	2.98	13.38	3.93	4.13	1.39
10	6.14	2.30	14.17	3.49	3.73	1.62
11	3.76	2.27	8.37	3.74	4.10	1.81
13	3.36	2.72	9.19	4.67	5.25	1.93
14	7.27	2.51	18.27	5.19	5.50	2.20
15	3.93	2.74	10.61	5.42	5.50	2.01
17	7.60	3.08	23.25	4.49	4.75	1.54
18	3.81	1.85	7.02	2.98	3.75	2.03
19	7.53	2.29	17.19	3.50	3.92	1.71
20	10.14	1.86	18.90	2.99	3.50	1.88
21	1.48	4.43	6.50	6.45	7.00	1.58
23	5.67	2.51	14.14	4.74	7.00	2.79
26	16.21	1.48	24.03	4.47	6.50	4.39

[a]In the cases where no descending branch was observed, Δ_u was taken as the maximum deformation executed in the test.

Table 4.9—Descriptions of instrumentation

Instrument No.[a]	Location	Data	Measurement type
SP01	Bottom north corner	Sliding of the panel	Displacement (in.)
SP02	Bottom north	Rocking of the panel	
SP03	Bottom north end connector	Uplift on the connector closest to north end	Displacement (in.)
SP04	Middle of the panel	Uplift at the middle of the panel	Displacement (in.)
SP05	Bottom south end connector	Uplift at the connector closest to south end	Displacement (in.)
SP06	Top north corner	Uplift at top CLT and wall	Displacement (in.)
SP07	Top south corner	Uplift at top CLT and wall	Displacement (in.)
SP08	Middle of the wall	Sliding at the middle of the panel	Displacement (in.)
SP09		Interpanel slip	Displacement (in.)
SP10		Interpanel slip	Displacement (in.)
SP11	Diagonal along the panel	Panel deformation, added after Test 7	Displacement (in.)
SP12	Diagonal along the panel	Panel deformation	Displacement (in.)
LC	Load cells	Used for boundary condition and also threaded rods from some tests	Load (kip)
SG	Strain gauges	Strain gauges on the threaded rods	Strain

[a]SP = string potentiometer, LC = load cell, SG = strain gauge.

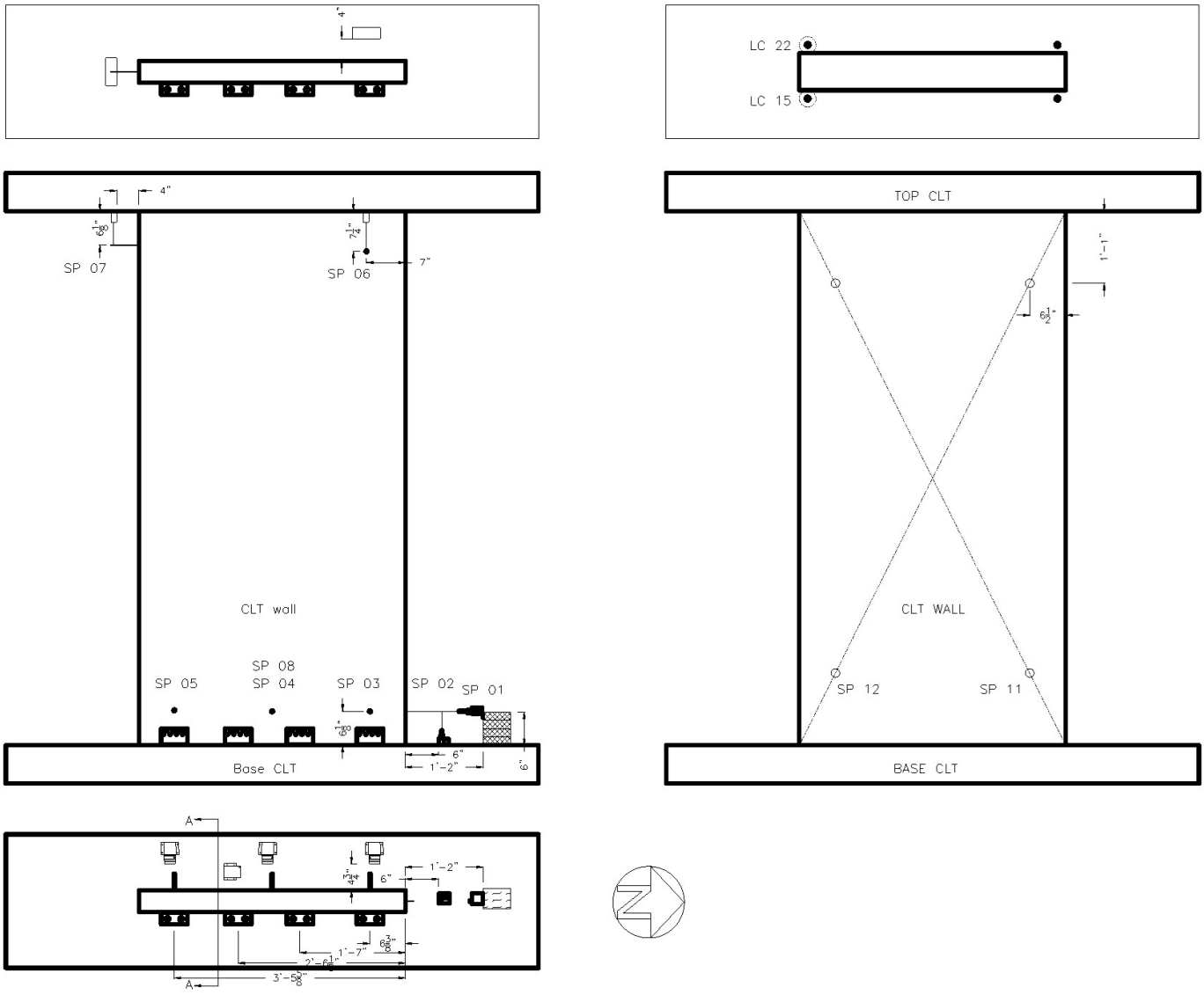


Figure 4.20. Single-panel configuration instrumentation (1 ft = 0.3 m; 1 in. = 25.4 mm).

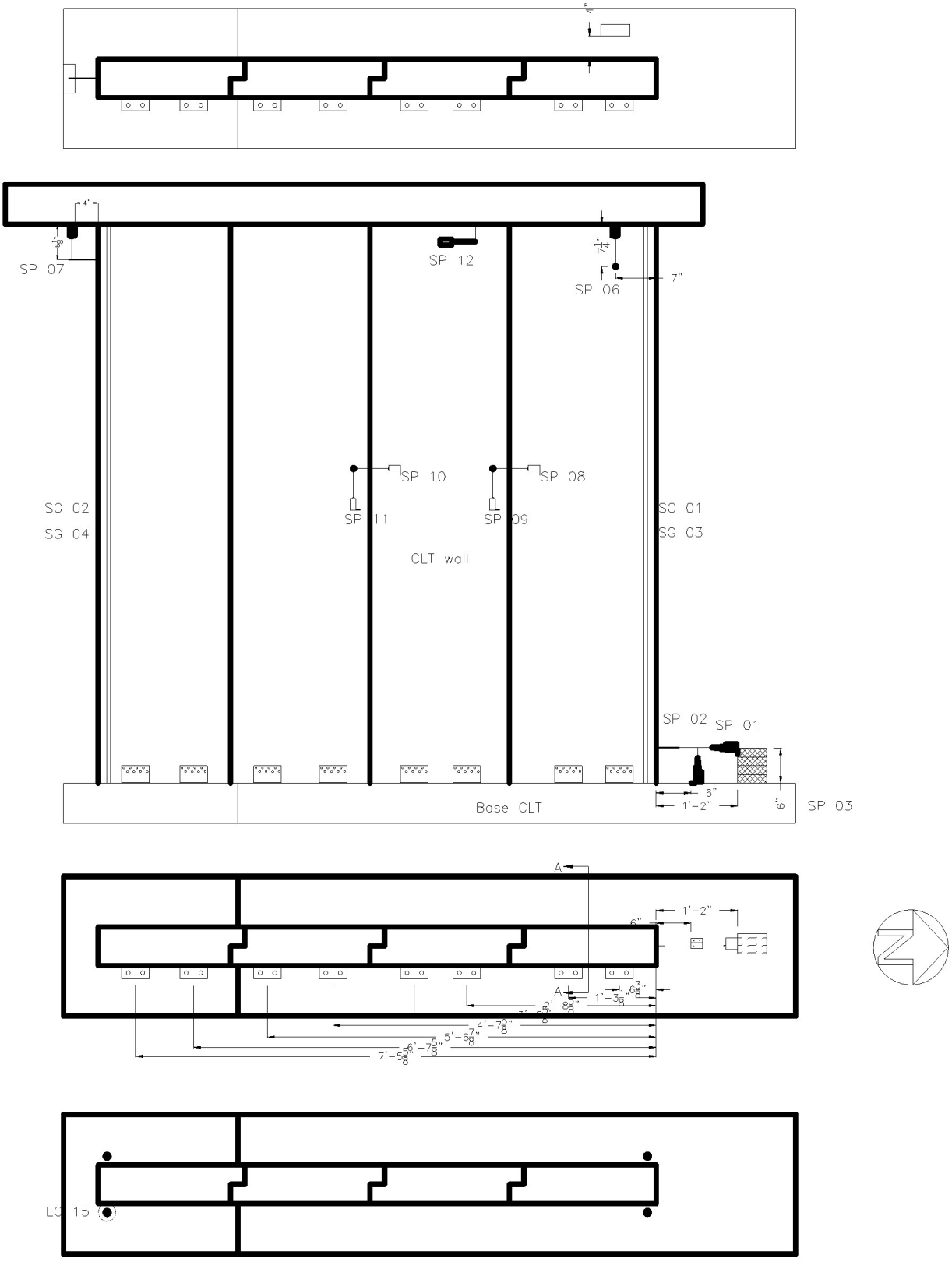


Figure 4.21. Multipanel configuration instrumentation (1 ft = 0.3 m; 1 in. = 25.4 mm).



Figure 4.22. Test 05: (a) three A3 connectors; (b) left connector; (c) middle connector; (d) right connector; (e) top corner of CLT panel; (f) bottom corner of CLT panel.

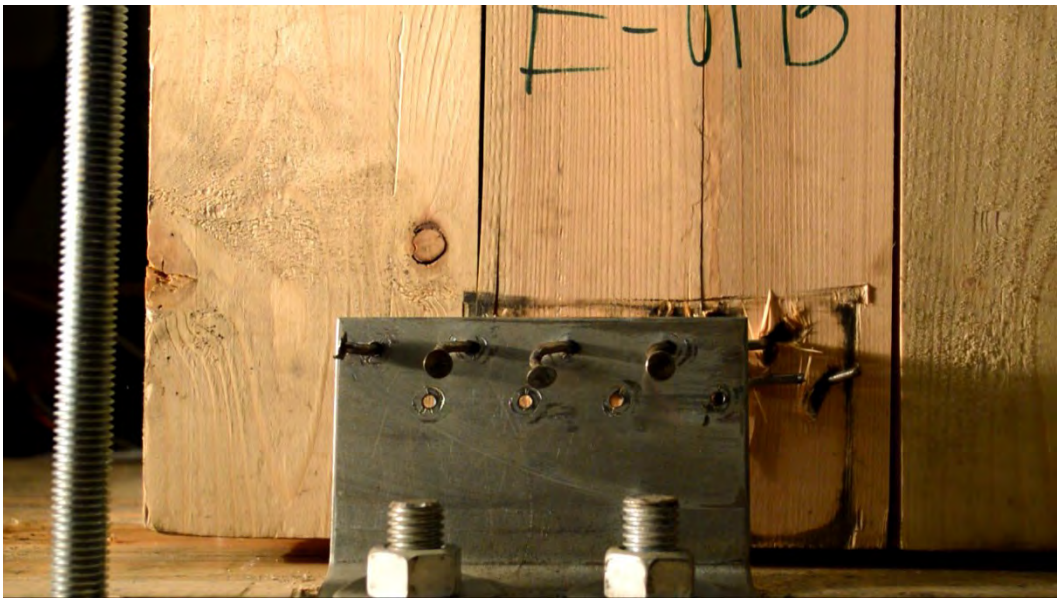


Figure 4.23. Test 03, three A3 connectors, edge connector.



Figure 4.24. Test 14, three A3 connectors, edge connector.

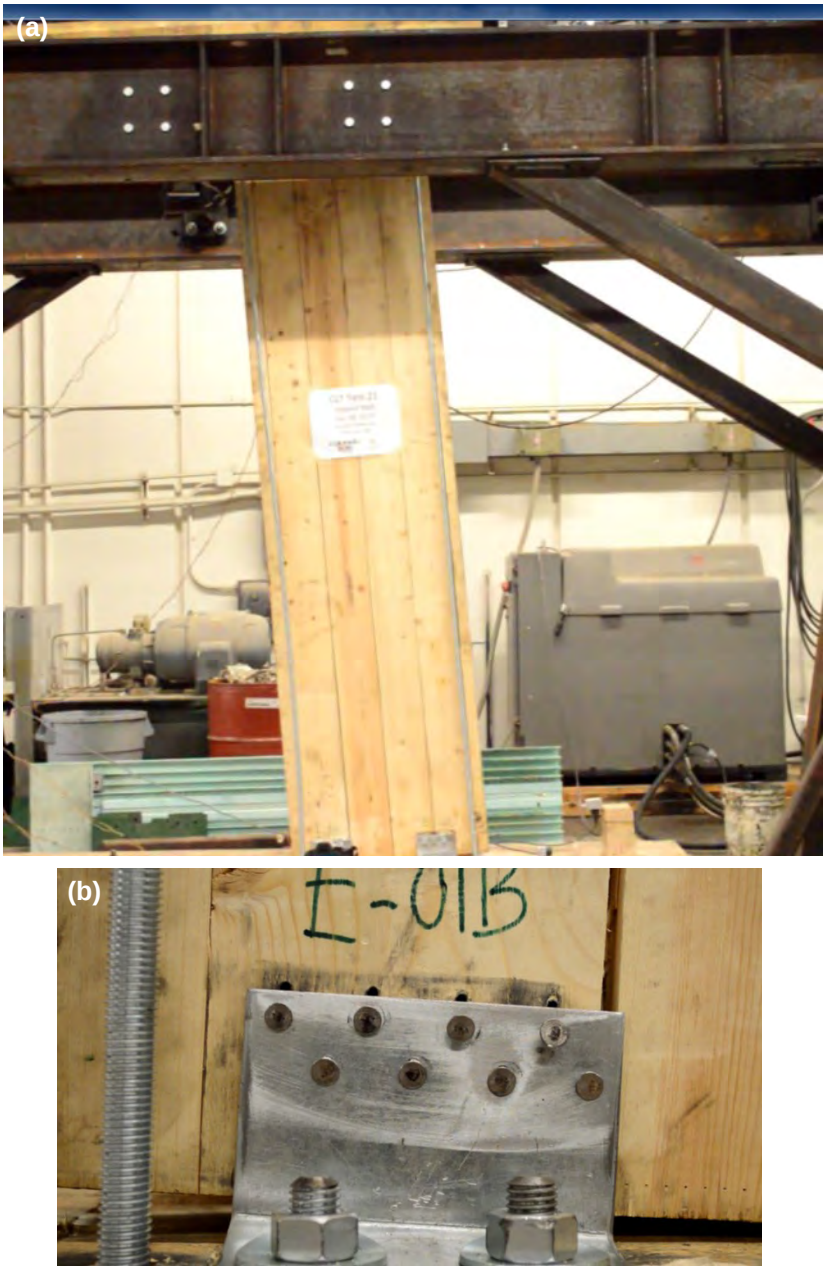


Figure 4.25. Test 21, 4:1 aspect ratio panel: (a) CLT panel rocking during test; (b) edge connector during rocking.

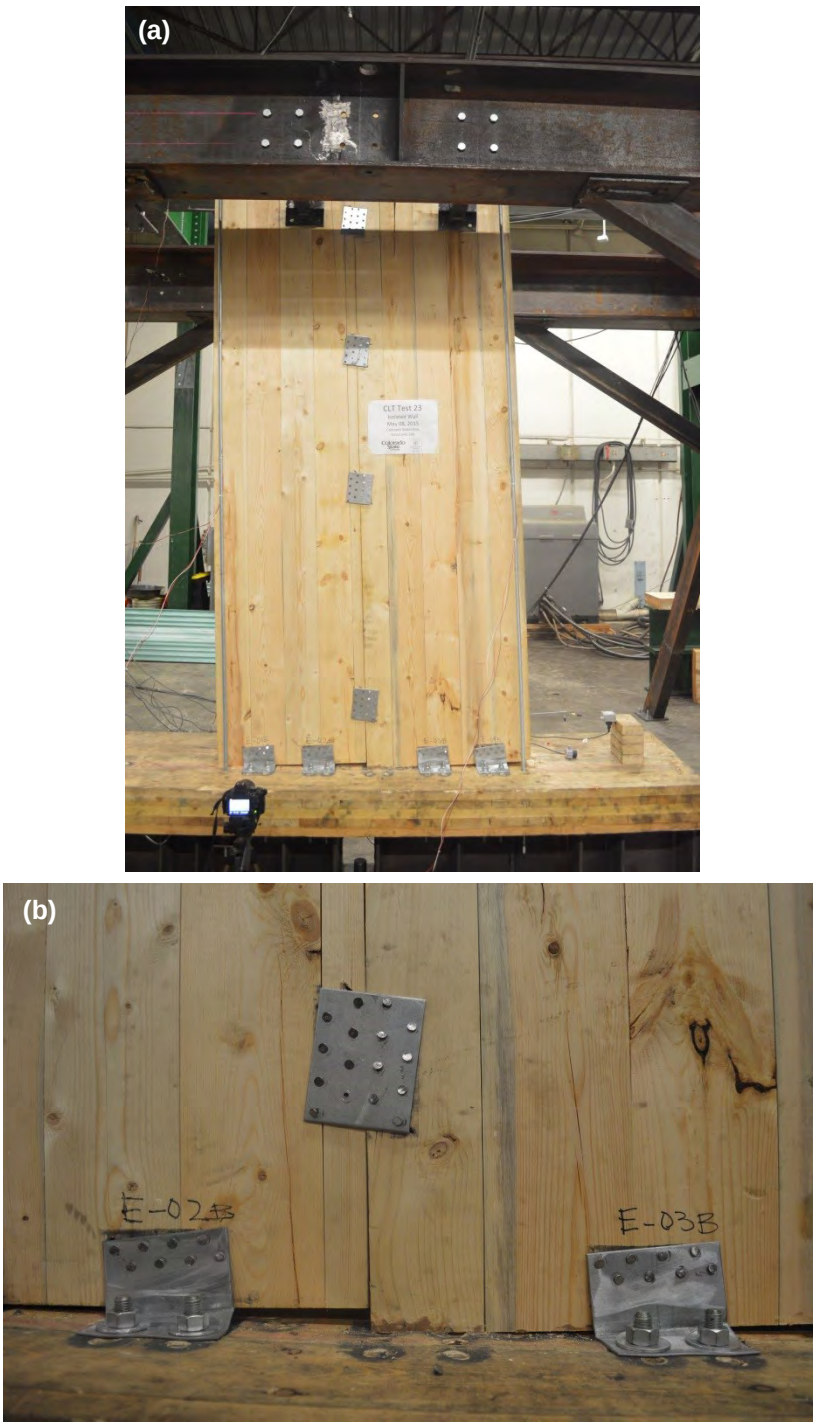


Figure 4.26. Test 23, two 2- by 8-ft (0.61- by 2.4-m) panels: (a) multipanel configuration rocking during the test; (b) edge connector and interpanel connectors.

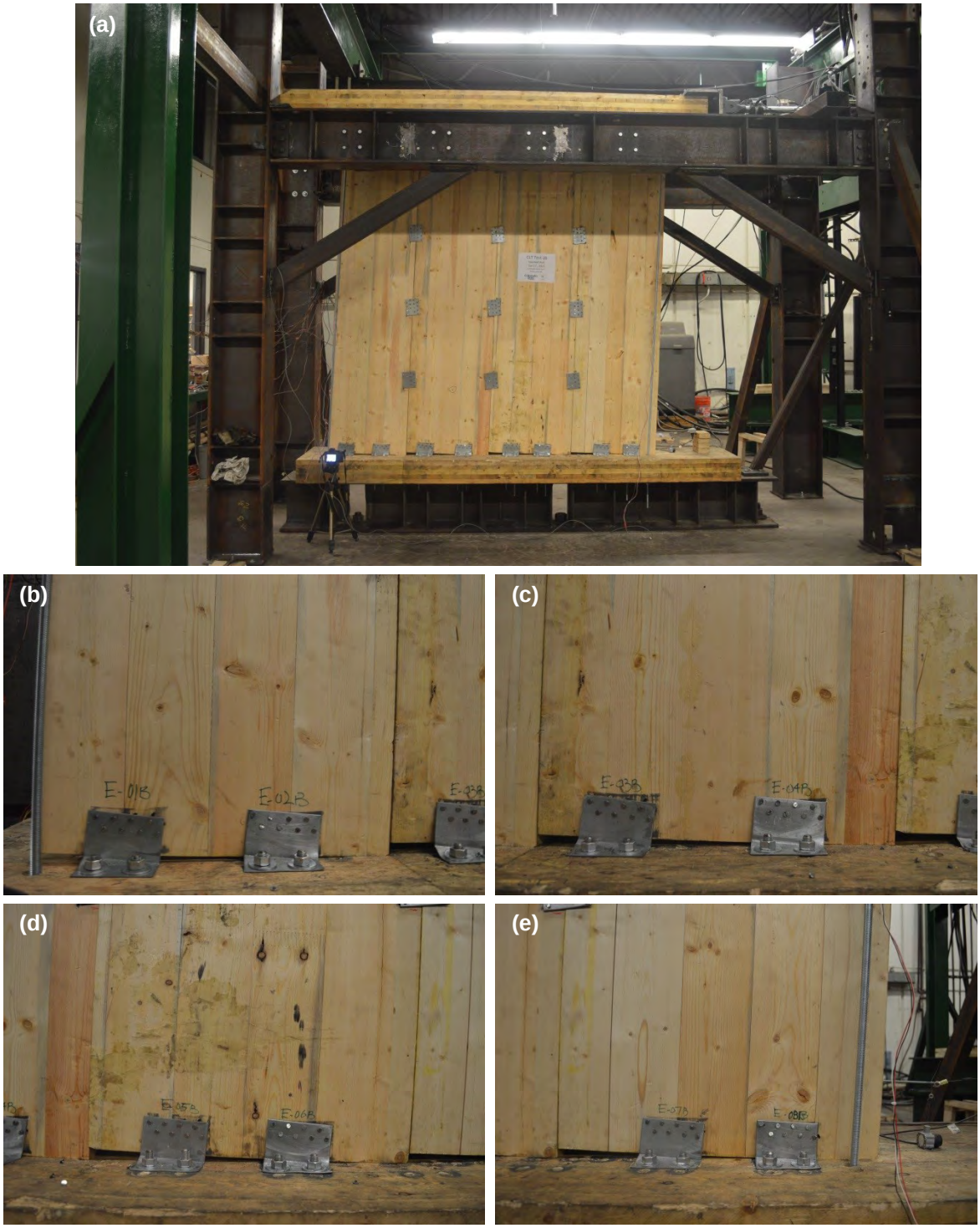
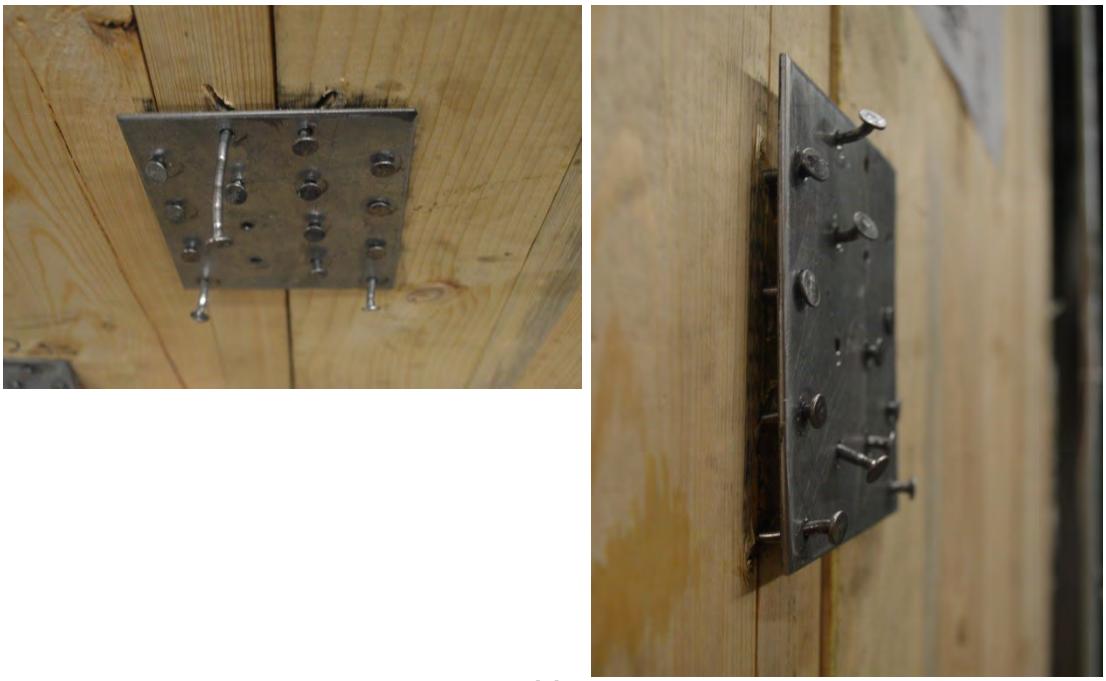
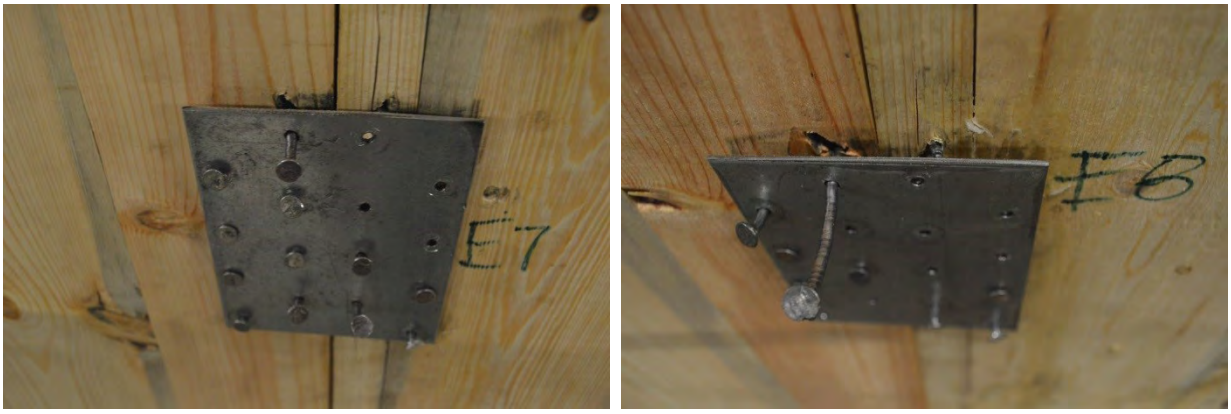


Figure 4.27. Test 26, four 2- by 8-ft (0.61- by 2.4-m) panels: (a) multipanel configuration rocking during the test; (b) first panel connectors; (c) second panel connectors; (d) third panel connectors; (e) fourth panel connectors.



(a)



(b)

Figure 4.28. Interpanel connectors: (a) Test 23; (b) Test 26.

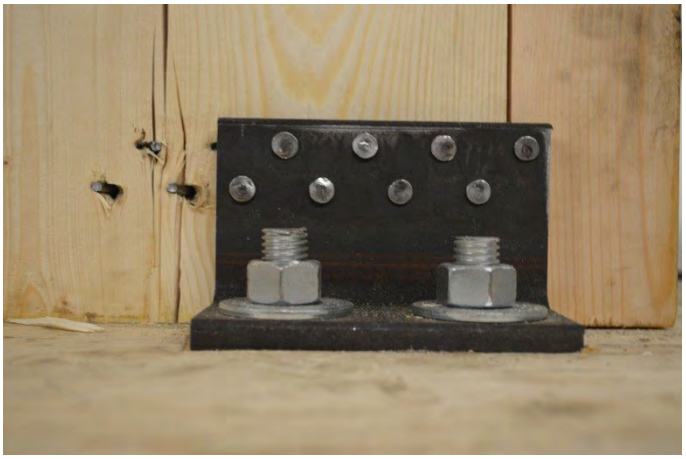


Figure 4.29. Al-type connector after test.

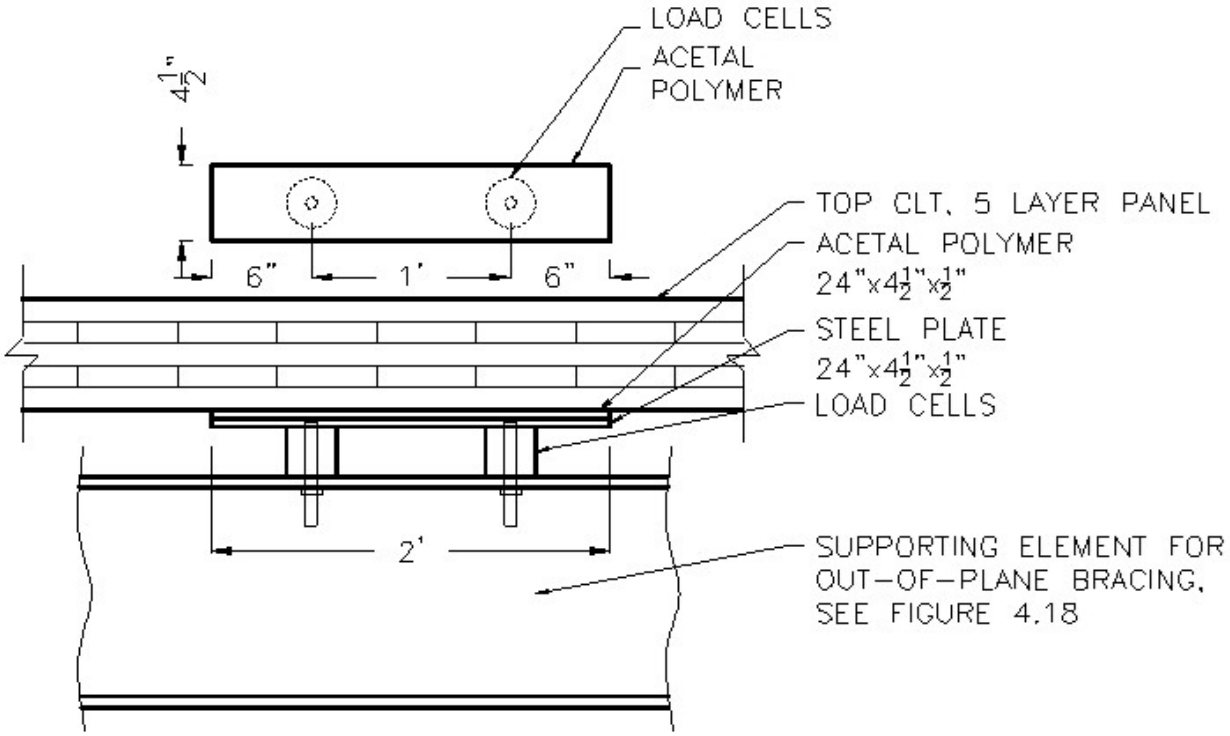




Figure 4.30. Floor diaphragm support (1 ft = 0.3 m; 1 in. = 25.4 mm).

Table 4.10—Friction test results

Test No.	Friction coefficient
1	0.330
2	0.281
3	0.280
4	0.210
5	0.387
6	0.317
7	0.337
8	0.289
9	0.292
10	0.271
Average	0.299
COV	0.157

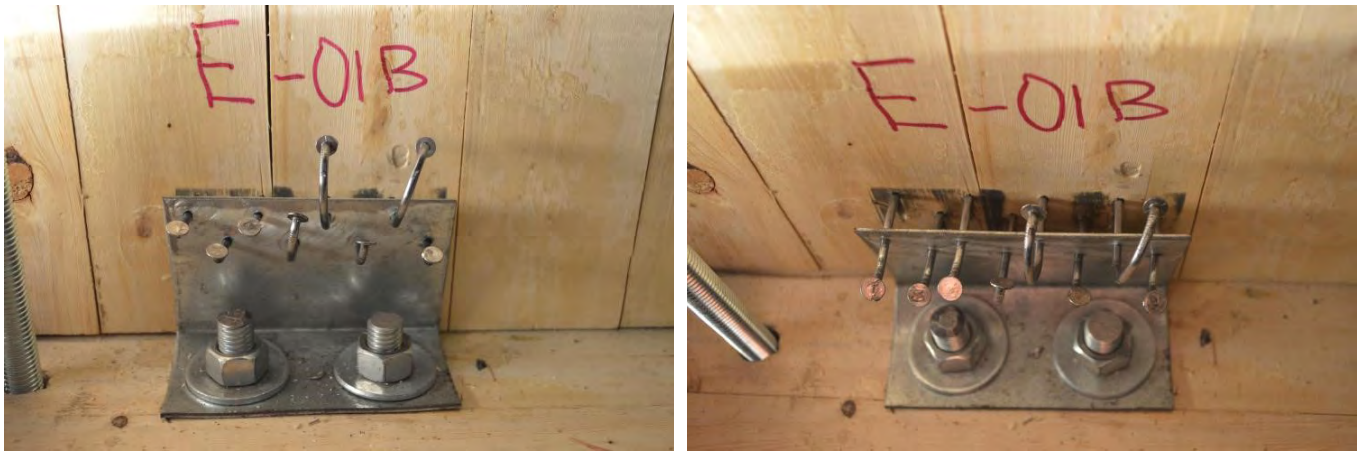


Figure 4.31. Connector failure CLT shear wall Test 05.

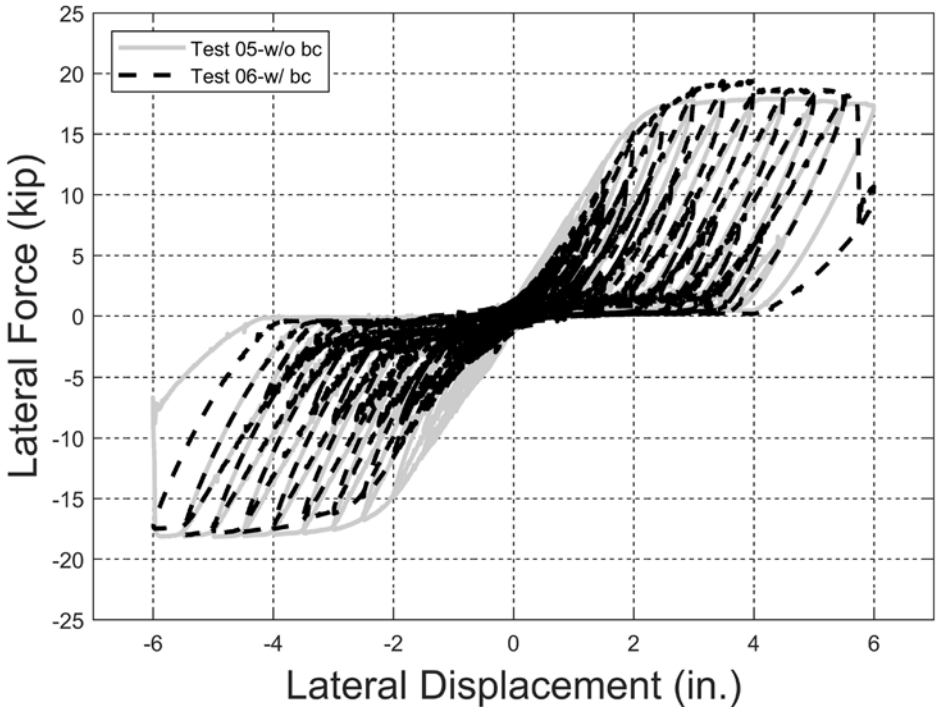


Figure 4.32. Hysteresis with and without boundary condition (bc) imposed (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

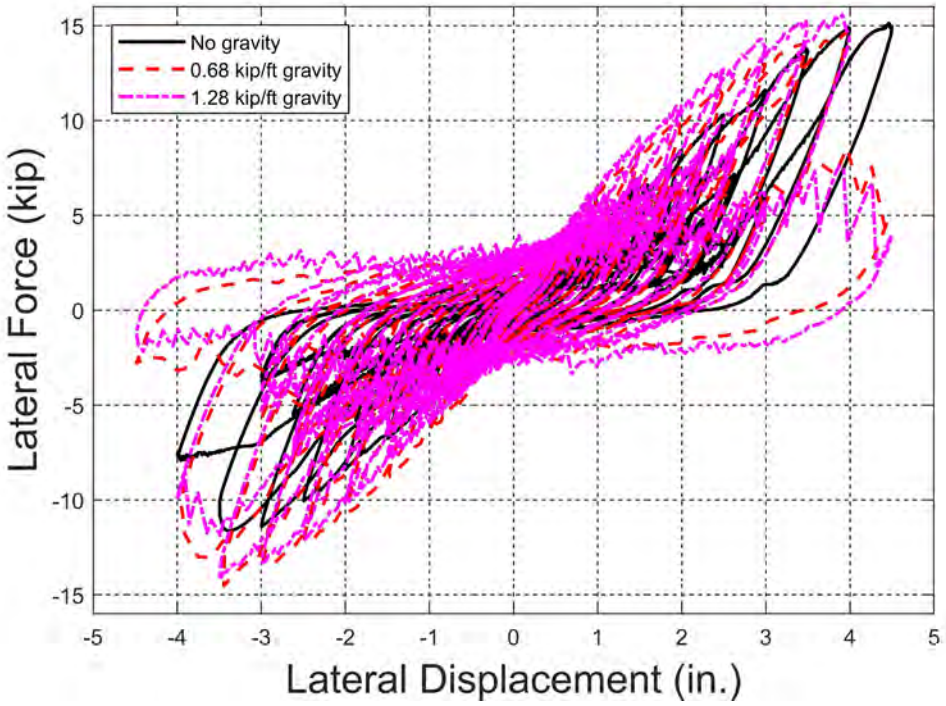


Figure 4.33. 4-ft by 8-ft by 6.65-in. (1.2-m by 2.4-m by 169-mm) panel tested under different vertical loading (data smoothing was performed) (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm; 1 ft = 0.3 m) .

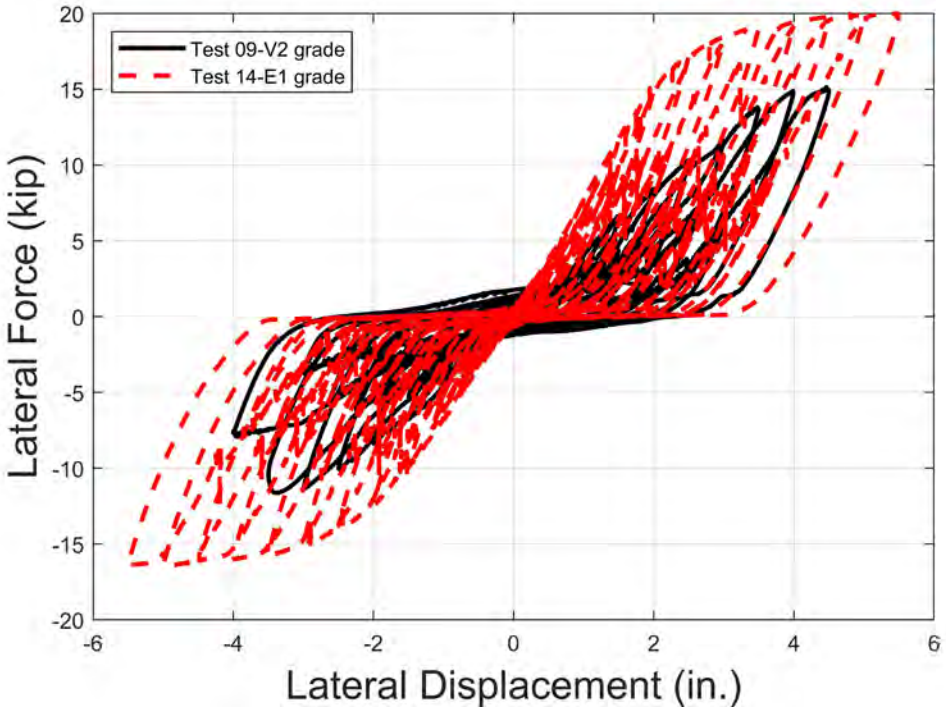


Figure 4.34. Hysteresis on tests of two different grades of CLT (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

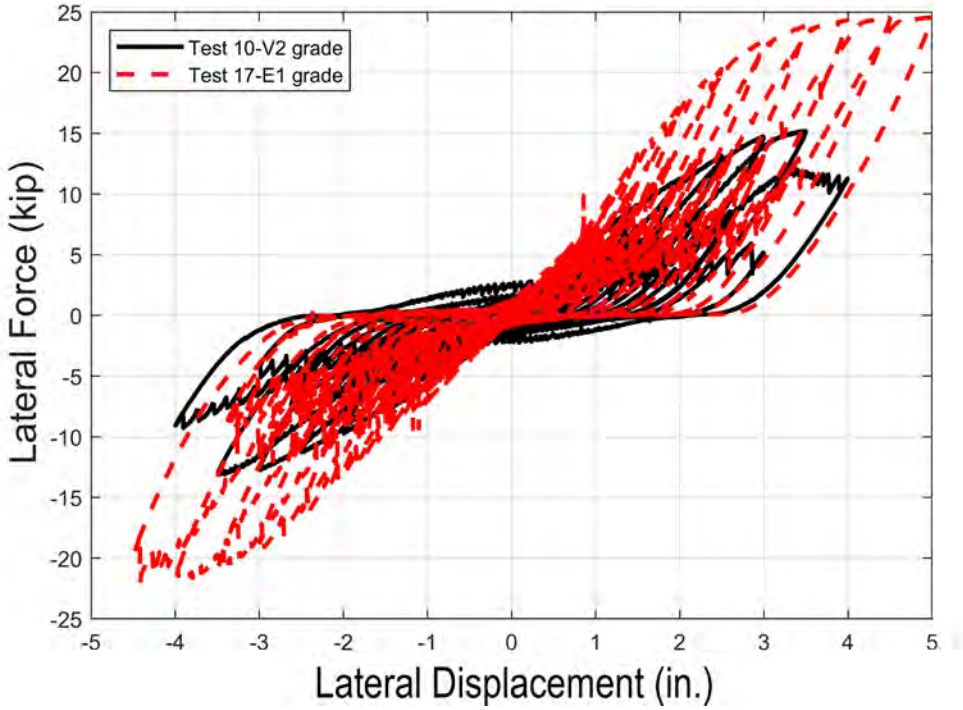


Figure 4.35. Hysteresis on tests of two different grades and thicknesses of CLT (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

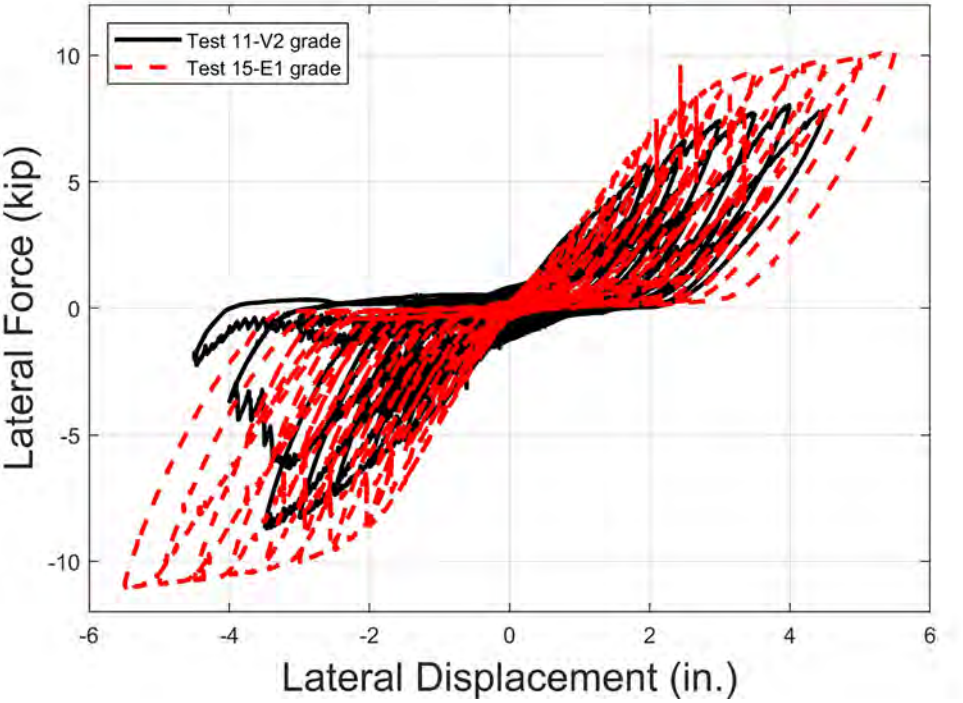


Figure 4.36. Hysteresis on tests of two different grades of CLT (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

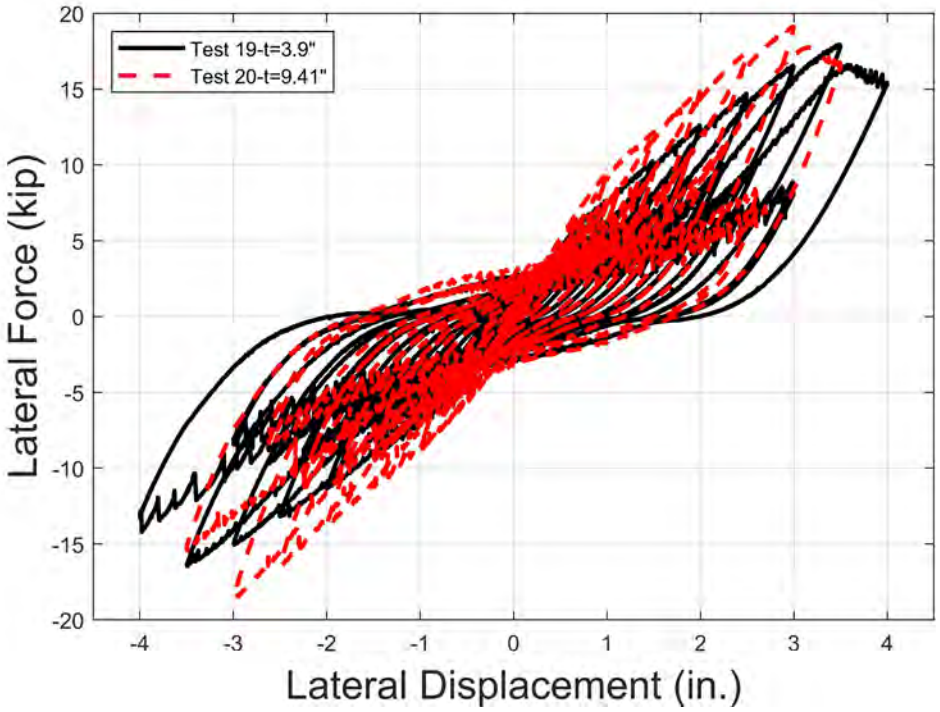


Figure 4.37. Hysteresis for different panel thicknesses (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

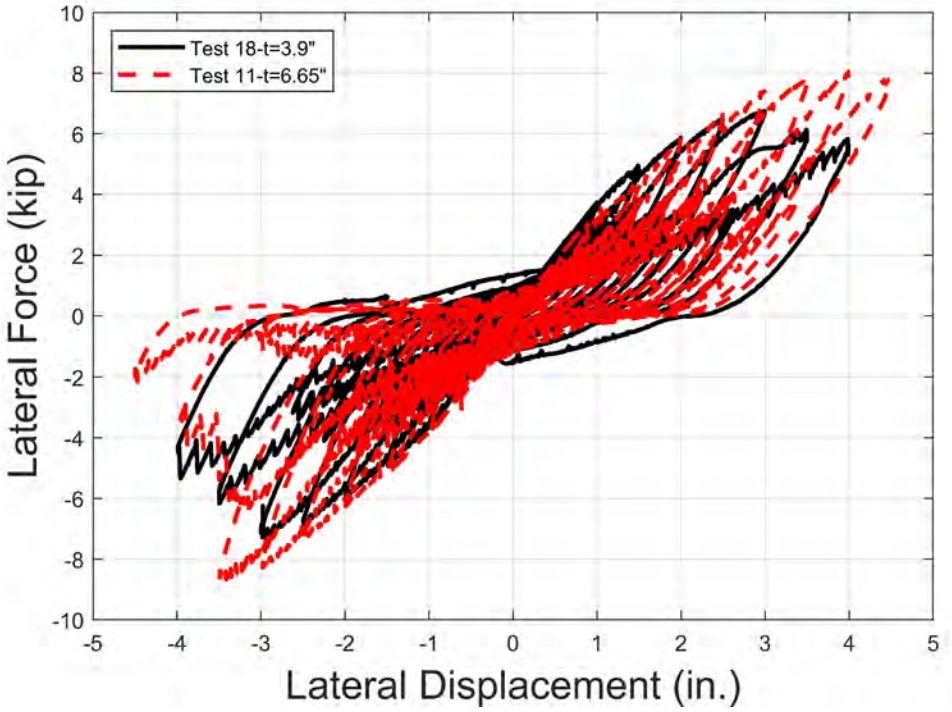


Figure 4.38. Hysteresis for different panel thicknesses (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

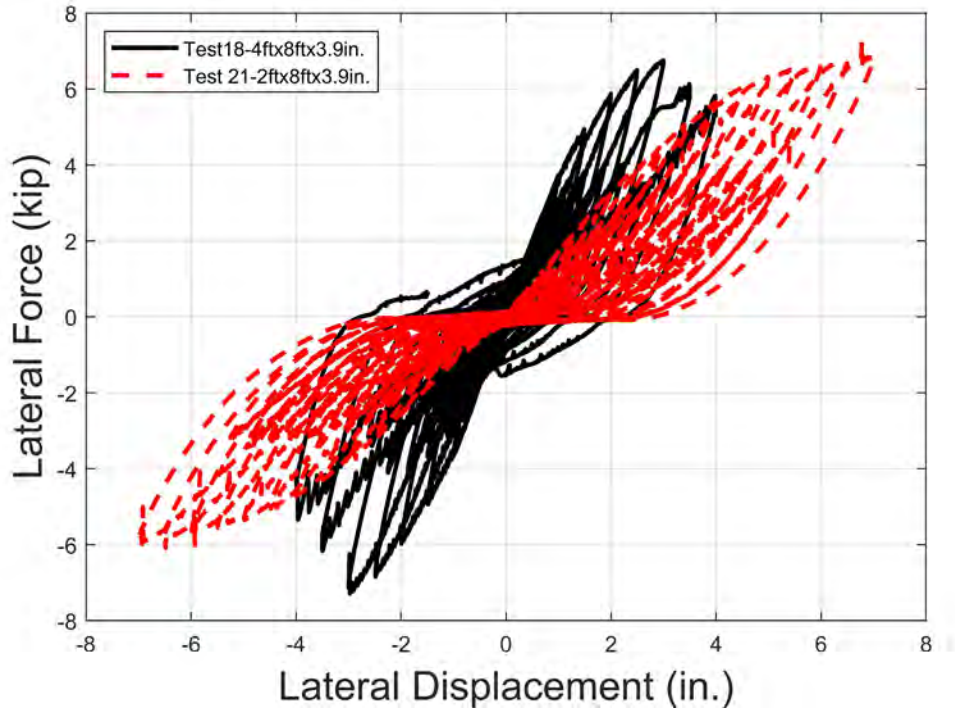


Figure 4.39. Hysteresis for 4:1 and 2:1 panel aspect ratio (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm; 1 ft = 0.3 m).

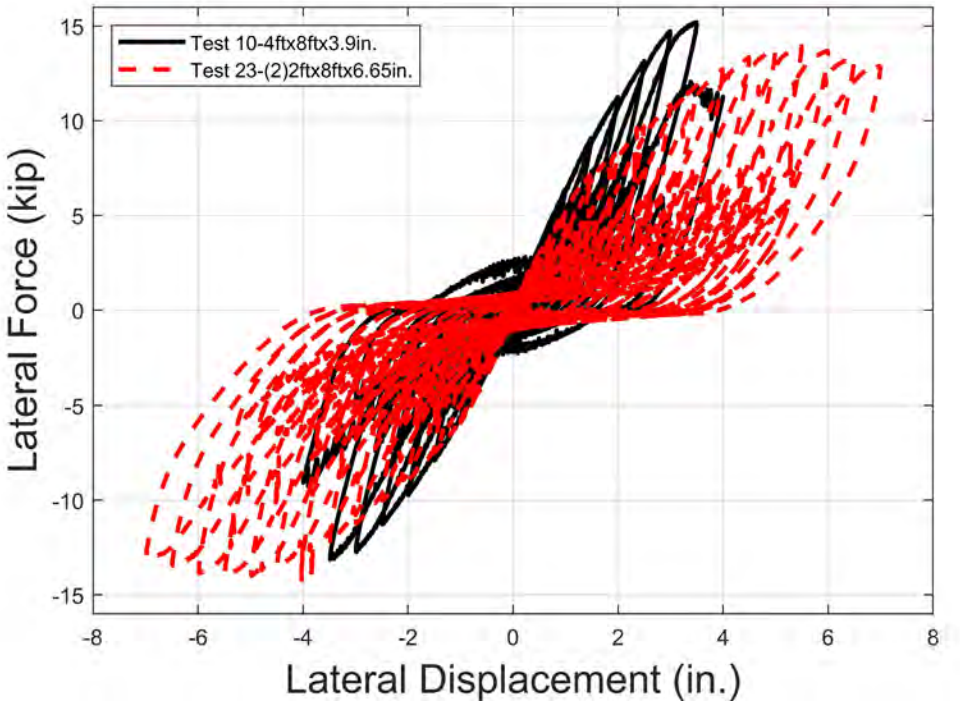


Figure 4.40. Hysteresis for cases with and without interpanel connector (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm; 1 ft = 0.3 m).



Figure 4.41. Test 23, shear wall with two 2-ft panels with vertical joint (1 ft = 0.3 m).



Figure 4.42. Test 26, shear wall with four 2-ft panels with vertical joint (1 ft = 0.3 m).

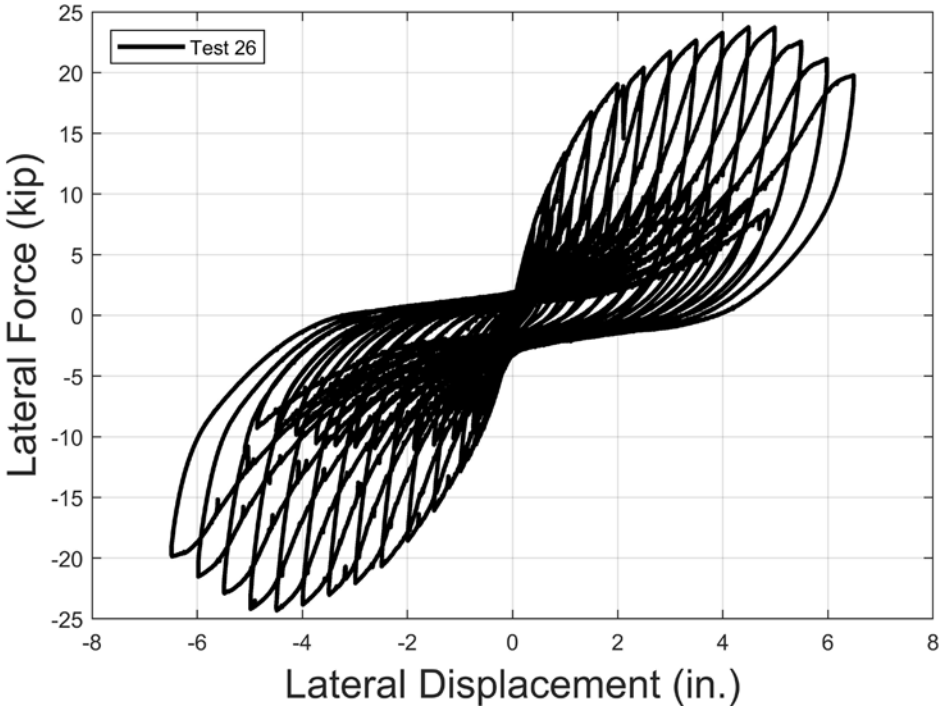


Figure 4.43. Test 26 hysteresis, shear wall with four 2-ft by 8-ft by 6.65-in. (0.6-m by 2.4-m by 169-mm) panels with vertical joint (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Chapter 5

Design Requirements for CLT Shear Wall Seismic Force Resisting Systems

5.1 Scope

These provisions shall be used for the design and construction of structural CLT members and connections that are part of a lateral force resisting system (LFRS). Capacity design principles are used to ensure development of the expected shear capacity of the prescribed nailed connectors of the CLT shear wall. The provisions provided herein shall be applied in combination with the requirements of the NDS, including appendix E, ASCE 7, Minimum Design Loads for Buildings and Other Structures, and the applicable building code.

5.1.1 Notation

The symbols used in this chapter have the following meaning:

C_G is CLT panel specific gravity adjustment factor

G is specific gravity

$GA_{eff\ (in\text{-}plane)}$ is effective in-plane shear stiffness of the CLT panel, lb/in of panel length

$EI_{eff\ (in\text{-}plane)}$ is effective in-plane bending stiffness of the CLT panel, lb-in^2

$V_{nail\ load}$ is load per nail, lb

h is CLT panel height, ft

b_s is individual CLT panel length, ft

Σb_s is sum of individual CLT panel lengths, ft

n is number of angle connectors along bottom of panel face

v_n is nominal unit shear capacity, lb/ft

v is unit shear induced by the design loads, lb/ft

Δ_a is vertical deformation of the wall hold-down system (including but not limited to fastener slip, device elongation, rod elongation, and uncompensated shrinkage plus the vertical compression deformation), the effects of which are measured at the ends of the shear wall and associated with induced unit shear in the shear wall, in.

$\Delta_{nail\ slip,h}$ is horizontal slip of nailed connections at top and bottom of panel, in.

$\Delta_{nail\ slip,v}$ is vertical slip of nailed connectors at vertical adjoining panel edge, in.

δ_{sw} is shear wall deflection, in.

ϕ_D is resistance factor for in-plane shear

5.1.2 Terminology

The terms listed below are to be used in addition to those in the Special Design Provisions for Wind and Seismic (SDPWS).

Cross-laminated timber shear wall: A shear wall employing CLT panels and meeting the requirements of this chapter.

Cross-laminated timber shear wall with shear resistance provided by high aspect ratio panels only: A CLT shear wall constructed with CLT panels of high aspect ratio in accordance with requirements of 5.3.7.

Platform construction: A method of construction by which roof and floor framing bear on load bearing walls that are not continuous through the story levels or floor framing.

5.2 Application Requirements

The design and construction of the CLT LFRS shall comply with all of the following:

1. The method of construction shall be platform construction in accordance with the following:

   a. CLT floor panels bear on and are supported by CLT shear walls that are part of the designated LFRS. Additional gravity support is permitted to be provided by other gravity framing system elements including but not limited to CLT walls that are not part of the designated LFRS, beams, columns, and light-frame walls.

   b. CLT floor panels are designed as the floor diaphragm to distribute lateral loads to the CLT shear walls.

2. CLT walls shall be classified as either (a) part of the designated LFRS (i.e., CLT shear wall) or (b) not part of the designated LFRS.

3. CLT walls that are not part of the designated LFRS shall meet the following requirements:

   a. CLT panels forming either a single-panel or multipanel wall shall have an aspect ratio, h/b_s, that is not less than the aspect ratio used for CLT shear wall panels in accordance with 5.3.1 or 5.3.7.

   b. Shear connections at the top and bottom of CLT wall panels shall be in accordance with 5.3.2. Where shear connections are provided at adjoining vertical panel edges to form a multipanel CLT wall, such connections shall be in accordance with 5.3.3. Hold-down systems in accordance with 5.3.4 are not required.

4. CLT walls that are not part of the designated seismic force resisting system (SFRS) shall be designed so that the action or failure of those elements will not impair the

vertical load and seismic force resisting capability of the designated SFRS.

a. The design shall provide for the effect of the CLT walls that are not part of the designated SFRS on the structural system at deformations corresponding to the design story drift, the distribution of forces to the structural system, and the corresponding load path to the final point of resistance. This shall be achieved by design for the most critical demands to vertical gravity load supporting elements, vertical SFRS elements, and diaphragms and their load paths, determined in accordance with both of the following:

i. Force and deformation demands determined including in the analysis CLT shear walls of the designated SFRS but neglecting in-plane shear strength and stiffness of CLT walls that are not part of the designated SFRS, and

ii. Force and deformation demands determined including in the analysis both CLT shear walls of the designated SFRS and CLT walls that are not part of the designated SFRS.

b. In addition, the effects of CLT walls classified as not part of the designated SFRS shall be considered where determining whether a structure has one or more of the irregularities defined in ASCE 7 12.3.2 based on the two analysis cases in accordance with item 4a.

c. Where the fundamental period of the structure is calculated in accordance with ASCE 7 using the structural properties and deformational characteristics, it shall include the effects of stiffness of CLT walls classified as not part of the designated SFRS.

d. For the purposes of Items 4a, 4b, and 4c, CLT wall panels that are classified as not part of the designated SFRS shall be modeled assuming they develop in-plane shear strength and stiffness associated with CLT shear walls of the same construction.

5. Shear distribution to individual CLT shear walls in a wall line shall provide the same calculated deflection in each shear wall (i.e., distribution of loads based on relative stiffness).

6. The dead load considered in the overturning design of each individual CLT panel within the CLT shear wall shall be limited to the dead load supported by or directly above the individual CLT panel.

7. CLT shear walls shall be full-height within each story. CLT shear walls are not permitted to be designed as Force-Transfer Around Openings (FTAO) shear walls or as Perforated Shear Walls.

8. The nominal unit shear capacity assigned to CLT shear walls shall not include contributions from connections other than those shear connections prescribed in 5.3.2 and 5.3.3.

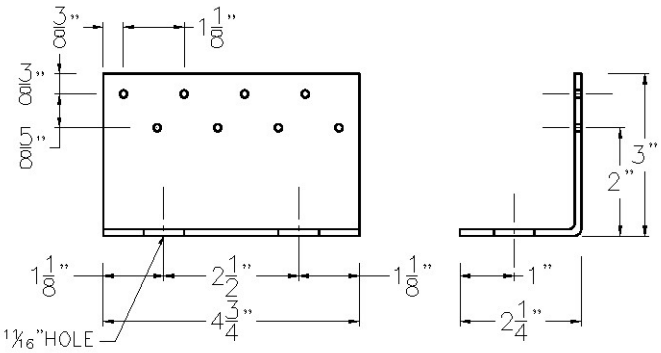


Figure 5.1. Top and bottom of wall angle connector (1 ft = 0.3 m; 1 in. = 25.4 mm).

9. ASCE 7 seismic criteria specific to light-frame construction shall not apply to the design of CLT shear walls and CLT diaphragms.

5.3 CLT Shear Wall Requirements

5.3.1 CLT Panels

CLT panels used in CLT shear walls shall be designed in accordance with the NDS and the following requirements:

1. CLT in-service moisture contents shall be less than 16% and specific gravity, G, shall be 0.35 or greater.

2. CLT panels forming either a single-panel or multipanel shear wall shall have aspect ratio, h/b_s, not greater than 4 nor less than 2. All CLT panels forming a multipanel shear wall shall have the same panel height, h, and individual panel length, b_s.

3. CLT panels shall be a minimum of 3.5 in. (88.9 mm) thick. Where angle connectors or vertical edge connectors are installed in both faces of the CLT panel and are directly opposed, CLT panels shall be a minimum of 6.875 in. (174.6 mm) thick so that nails from opposing faces do not overlap.

4. Holes, notches, and other modifications to CLT panels shall not be permitted unless the effects of removal of material on load carrying capacity of the CLT panel is determined by an approved rational analysis.

5.3.2 Top- and Bottom-of-Wall Angle Connectors

Shear connections at the top and bottom of CLT shear walls shall be composed of prescribed steel angle connectors, nails, and bolts in accordance with the following requirements:

1. Angle connectors shall be fabricated from a 0.105-in.-(2.7-mm-) thick ASTM A653 Grade 33 steel sheet with the geometry as illustrated in Figure 5.1.

2. Vertical legs of angle connectors shall be fastened to wall panel using eight 16d carbon steel box nails (3.5-in.-(88.9-mm-) long by 0.135-in. (3.4-mm) shank diameter by 0.344-in. (8.7-mm) head diameter) with bright finish in

accordance with ASTM F1667 (ASTM 2018b) including Supplementary Requirements of ASTM F1667 S1 Nail Bending Yield Strength.

3. Horizontal legs of angle connectors shall be fastened to supporting elements (e.g., CLT floor or roof panels, concrete foundation elements, or roof framing elements other than CLT) with two 5/8-in.- (15.9-mm-) diameter by 4-1/2-in.- (114.3-mm-) long (minimum) bolts to provide minimum 4-1/2-in. (114.3-mm) bearing length with washer per American Society of Mechanical Engineers (ASME) B18.21.1, or two 5/8-in. (15.9-mm) full-body-diameter lag screws with 2-3/4-in. (69.9-mm) thread penetration (minimum) excluding tapered tip and 3-1/8-in. (79.4-mm) unthreaded shank length (minimum) to provide minimum 5-7/8-in. (149.2-mm) bearing length and with washer per ASME B18.21.1 (ASME 2016). Bolts and lag screws shall be ASTM A307 (ASTM 2014a) Grade A or higher. The anchorage, foundation, and other supporting elements shall be capable of resisting a concurrent tension force and shear force transmitted through horizontal leg fasteners, with force in each direction equal to the connector nominal shear capacity. The design of the 5/8-in.- (15.9-mm) diameter anchor to concrete foundation or other concrete elements shall be in accordance with American Concrete Institute (ACI) 318 (ACI 2019) and the prescribed loading above shall be considered to meet the ductile yield mechanism requirement of ACI 318.

4. Angle connectors at the bottom and top of wall panels shall extend to within 12 in. (0.31 m) of each end of each panel of a single-panel or multipanel shear wall.

5. Each wall panel shall have at least two angle connectors at the top and bottom. The same number of angle connectors shall be provided at the top and bottom of each wall panel.

5.3.3 Adjoining Panel Edge Connector

In CLT multipanel shear walls, shear connections at adjoining vertical panel edges of CLT wall panels shall be composed of prescribed steel plate connectors and nails in accordance with the following requirements:

1. Plate connector shall be fabricated from a 0.105-in.- (2.7-mm-) thick ASTM A653 Grade 33 steel sheet with the geometry as illustrated in Figure 5.2.

2. Plate connectors shall be fastened to each wall panel using eight 16d carbon steel box nails (3.5-in.-long by 0.135-in. shank diameter by 0.344-in. head diameter) with bright finish in accordance with ASTM F1667 including Supplementary Requirements of ASTM F1667 S1 Nail Bending Yield Strength.

3. The number of plate connectors required at adjoining vertical edges of CLT panels shall equal the number of angle connectors along the bottom edge of the wall panel times the CLT panel aspect ratio, h/b_s, rounded up to the next whole number.

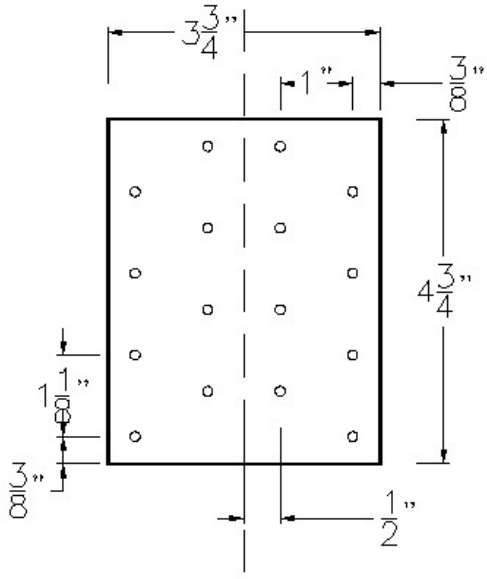


Figure 5.2. Adjoining panel edge connector (1 ft = 0.3 m; 1 in. = 25.4 mm).

5.3.4 Hold-Down System

Each end of each shear wall shall be provided with a hold-down system. Hold-down systems shall comply with the following:

1. Hold-down systems shall consist of continuous tie-rod systems or conventional hold-down devices that use threaded anchor rods and nail, screw, or bolt attachment to the CLT panel.

2. Where continuous tie-rod systems are used, rods at each level shall be designed for cumulative overturning tensile forces and bearing plates shall be provided at the floor level above each story. Tie-rod elongation or conventional hold-down device deformation for each story shall not exceed 0.185 in. (4.7 mm) when applying strength design load combinations of ASCE 7.

3. The hold-down system including anchorage to the foundation shall be designed to resist not less than 2.0 times the forces associated with the design shear capacity of the CLT shear wall determined in accordance with NDS 4.1.4.1 for seismic and 1.5 times the forces associated with the design shear capacity of the CLT shear wall in accordance with NDS 4.1.4.2 for wind. Connections between the hold-down device and the CLT panel shall comply with net section tension rupture, row tear-out, group tear-out in accordance with NDS appendix E. The anchorage to foundation shall be designed in accordance with ACI 318.

5.3.5 Compression Zone

The length of the compression zone for overturning compression forces induced by the design loads shall be determined to satisfy static equilibrium assuming a uniform distribution of bearing stress in the compression zone. For multipanel shear walls, the compression zone shall be

contained within the outermost wall panel. CLT wall panel resistance to induced axial compression forces shall be determined using cross section dimensions associated with the compression zone.

5.3.6 Other Load Path Connections to CLT Wall Panels

Connections to CLT wall panels in addition to those connections prescribed in 5.3.2 and 5.3.3 shall be provided in accordance with NDS 4.1.1 to provide a continuous load path. Load path connections to CLT wall panels shall be with dowel-type fasteners designed to develop Mode IIIs or Mode IV yielding and shall comply with net section tension rupture, row tear-out, group tear-out in accordance with NDS appendix E. Screws (e.g., wood screws and lag screws) shall not be used in supplemental top and bottom of wall connections to supporting elements. Angle connectors prescribed in 5.3.2 shall not be considered in design to resist out-of-plane forces.

5.3.7 CLT Shear Walls with Shear Resistance Provided by High Aspect Ratio Panels Only

CLT shear walls with shear resistance provided by high aspect ratio panels only shall meet all requirements applicable for CLT shear walls and the following:

a) all CLT wall panels used as part of the designated LFRS shall have aspect ratio, h/b_s, equal to 4, and

b) all CLT wall panels that are not part of the designated LFRS shall have aspect ratio, h/b_s, not less than 4.

5.4 CLT Shear Wall Deflection

CLT shear wall deflection, δ_{SW}, shall be calculated by use of the following equation:

$$\delta_{SW} = \frac{576 v b_s h^3}{EI_{eff\ (in\text{-}plane)}} + \frac{vh}{GA_{eff\ (in\text{-}plane)}} + 3\Delta_{nail\ slip,\ h} + 2\Delta_{nail\ slip,\ v}\frac{h}{b_s} + \Delta_a \frac{h}{\Sigma b_s} \quad (5.1)$$

where

v is unit shear induced by the design loads, lb/ft,

$EI_{eff\ (in\text{-}plane)}$ is effective in-plane bending stiffness of the of CLT panel, lb-in^2,

$GA_{eff\ (in\text{-}plane)}$ is effective in-plane shear stiffness of the CLT panel, lb/in of panel length,

h is CLT panel height, ft,

b_s is individual CLT panel length, ft,

Σb_s is sum of individual CLT panel lengths, ft,

$\Delta_{nail\ slip,h} = V_{nail\ load}/6{,}700$, in.,

$\Delta_{nail\ slip,v} = \Delta_{nail\ slip,h}$, in. (= 0 for a single-panel shear wall),

$V_{nail\ load}$ is load per nail, lb (calculated as total shear load at base of wall divided by total number of nails in base connectors), and

Δ_a is vertical deformation of the wall hold-down system (including but not limited to fastener slip, device elongation, rod elongation, and uncompensated shrinkage plus the vertical compression deformation), the effects of which are measured at the ends of the shear wall and associated with unit shear induced by the design loads in the shear wall, in.

5.5 Nominal Unit Shear Capacity

Nominal unit shear capacity, v_s, shall be in accordance with Equation 5.2. Where both faces of a panel are provided with angle connectors in accordance with 5.3.2, the nominal unit shear capacity shall be permitted to be taken as the sum of the nominal unit shear capacities of each face:

$$v_s = n\left(\frac{2{,}605}{b_s}\right)(C_G) \quad (5.2)$$

where

n is number of angle connectors along bottom of panel face,

2,605 = connector nominal shear capacity in accordance with NDS (lb),

b_s = individual CLT panel length (ft), and

C_G = CLT panel specific gravity adjustment factor. $C_G = 1.0$ for $G \geq 0.42$, and $C_G = 0.86$ for $G = 0.35$. Linear interpolation shall be permitted to be used to determine values of C_G for G between 0.35 and 0.42.

5.6 ASD and LRFD Design Unit Shear Capacities

For seismic design, the load and resistance factor design (LRFD) factored unit shear resistance shall be determined by multiplying the nominal unit shear capacity by a resistance factor, ϕ_D, of 0.50. For seismic design, the allowable stress design (ASD) allowable unit shear capacity shall be determined by dividing the nominal unit shear capacity by the ASD reduction factor of 2.8.

5.7 Diaphragm Requirements

CLT floor diaphragms shall be designed in accordance with principles of mechanics using values of fastener and member strength in accordance with NDS. Fasteners used in floor panel joints and connection of the diaphragm chord shall be in accordance with NDS and have sufficient embedment to develop Mode IIIs or Mode IV yielding. In addition, the diaphragm chord and its connections shall be designed such that their nominal strengths exceed forces associated with development of the diaphragm nominal unit shear capacity. Fasteners used in floor panel joints, such as lap joints and spline joints, shall not be used to meet requirements for continuity of diaphragm tension chords. Where steel splice plates are used to form the tension chords, the steel shall be sized to have design yield strength

not less than 2 times the design forces, either ASD or LRFD as applicable, to protect against concentration of yielding in the steel splice prior to development of connection strength between the steel and wood.

Special design force and detailing provisions for anchorage of concrete–masonry structural walls to diaphragms of ASCE 7 section 12.11 are applicable for the design of CLT diaphragms. Wood structural panel splines used for shear transfer in CLT diaphragms shall not be used to provide continuous cross ties required by section 12.11.

CLT diaphragm deflection shall be determined using established principles of engineering mechanics.

A summary of the CLT shear wall tests and a comparison with the design methodology are provided in Table 5.1. Commentary on the design methodology is provided in Appendix D, typical detailing is provided in Appendix E, and an example of floor bearing design for overturning is discussed in Appendix F.

Table 5.1—Summary of CLT isolated shear wall tests (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton; 1 lb = 4.45 N)

										Strength design						Drift design		
									Hold-down rod (No. and diam. in in.)	F_{max} (kip)		LRFD[a] nominal capacity (kip)	LRFD design strength (kip)	Test/ LRFD design strength[b]	Load in test wall (lb)	Min. measured total deflection (in.)[c]	Theoretical deflection (in.)	Measured/ theory
Test No.	Grade	Height (ft)	Length (ft)	No. of plys	Thickness (in.)	No. of connectors	Connector type	Gravity (kip/ft)		+ve	–ve							
3	V2	8	4	5	6.65	3	A3	0.68	(2) 5/8	14.79	14.62	7.8	3.91	3.74	3,907.5	0.48	0.63	0.76
4	V2	8	4	5	6.65	3	A3	1.28	(2) 5/8	15.7	14.22	7.8	3.91	3.64	3,907.5	0.50	0.59	0.85
5	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8	17.9	18.15	7.8	3.91	4.58	3,907.5	0.36	0.55	0.65
6	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8	19.51	18.02	7.8	3.91	4.61	3,907.5	0.51	0.55	0.93
9	V2	8	4	5	6.65	3	A3	—	(2) 5/8	15.11	11.62	7.8	3.91	2.97	3,907.5	0.87	0.67	1.30
10	V2	8	4	3	3.9	4	A3	—	(2) 5/8	15.2	13.12	10.4	5.21	2.52	5,210.0	0.81	0.90	0.90
11	V2	8	4	5	6.65	2	A3	—	(1) 5/8	8.03	8.7	5.2	2.61	3.08	2,605.0	0.63	0.70	0.90
13	E1	8	4	5	6.89	2	A3	—	(1) 5/8	7.597	10.72	5.2	2.61	2.91	2,605.0	0.68	0.63	1.08
14	E1	8	4	5	6.89	3	A3	—	(2) 5/8	19.98	16.52	7.8	3.91	4.23	3,907.5	0.51	0.59	0.86
15	E1	8	4	5	6.89	2	A3	—	(1) 5/8	10.16	11.02	5.2	2.61	3.89	2,605.0	0.58	0.63	0.92
17	E1	8	4	5	6.89	4	A3	—	(2) 5/8	24.5	21.9	10.4	5.21	4.20	5,210.0	0.64	0.67	0.96
18	V2	8	4	3	3.9	2	A3	—	(1) 5/8	6.74	7.3	5.2	2.61	2.58	2,605.0	0.67	0.77	0.87
19	V2	8	4	3	3.9	5	A3	—	(2) 3/4	17.93	16.43	13.0	6.51	2.52	6,512.5	0.78	0.92	0.85
20	V2	8	4	7	9.41	5	A3	—	(2) 3/4	19.11	18.69	13.0	6.51	2.87	6,512.5	0.61	0.70	0.87
21	V2	8	2	3	3.9	2	A3	—	(2) 5/8	7.19	5.95	5.2	2.61	2.28	2,605.0	1.55	1.90	0.82
23	V2	8	2 (2)	5	6.65	4	A3	—	(2) 5/8	13.34	13.84	10.4	5.21	2.56	5,210.0	0.84	1.05	0.80
26	V2	8	4 (2)	5	6.65	8	A3	—	(2) 5/8	23.72	24.3	20.8	10.42	2.28	10,420.0	0.67	0.73	0.92
																	Avg.	0.90

[a]LRFD, load and resistance factor design.
[b]The ratio was calculated based on the maximum of the positive and negative excursion.
[c]Minimum of the positive and negative excursion.

Chapter 6

Development of Numerical Model and Collapse Modes

Nonlinear static and dynamic analyses were performed on the index archetypes that meet all requirements specified within the methodology. Static pushover analysis was performed to determine the period-based ductility (μ_T) and overstrength factors (Ω). Incremental dynamic analysis (IDA) was performed on all the archetypes for a set of predefined earthquakes termed "far-field" earthquakes scaled as MCE. The results of the IDA were used to plot the cumulative distribution function (CDF), which then led to the determination of collapse spectral acceleration (S_{CT}).

6.1 Numerical Modeling

CLT modeling can be performed with various levels of complexity, and although a finite element (FE) based formulation can be used in certain cases, the computational effort for a study such as FEMA P695 typically demands the use of models with simplified kinematic assumptions. Therefore, analysis was performed using the SAPWood software (developed by CSU, Fort Collins, Colorado, USA) (Pei and van de Lindt 2007), which was developed as part of the NEESWood project (NEES, network for earthquake engineering) for analysis for light-frame wood structures. The software is based on the shear-bending coupled model, and the assumptions involved in this model are as follows:

- floor diaphragm can be represented as rigid plates with six degrees of freedom,
- shear resistance can be represented using hysteretic springs,
- overturning restraint can be represented using multilinear springs, and
- effect of finish materials was ignored for now (may be added).

The accuracy and reliability of the software has been validated through a number of studies using full-scale system-level test data (Pei and van de Lindt 2009, van de Lindt and others 2010). A simplified representation of the analytical model in SAPWood is shown in Figure 6.1. It is important to note that although SAPWood is capable of performing 3D analysis, the archetypes analyzed in this study were simplified 2D models.

According to FEMA P695 methodology, to the extent possible, the numerical model used for components should simulate all significant deterioration mechanisms that can lead to collapse, i.e., degradation in stiffness and strength and inelastic deformation. Based on the test results, CLT shear walls with connectors used as energy dissipation mechanisms exhibit hysteretic behavior similar to light-frame wood shear walls. The phenomenological CUREE hysteretic model, developed by Folz and Filiatrault (2001)

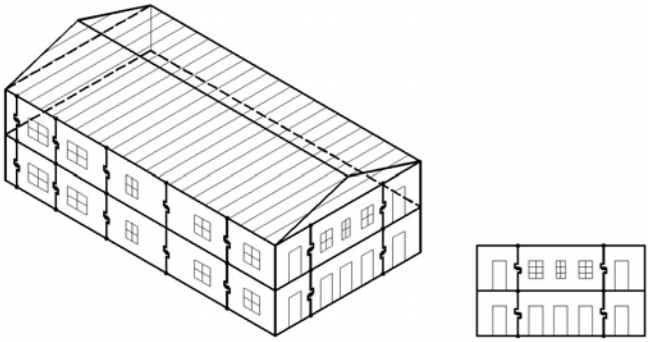


Figure 6.1. Analysis model and simplified 2D model.

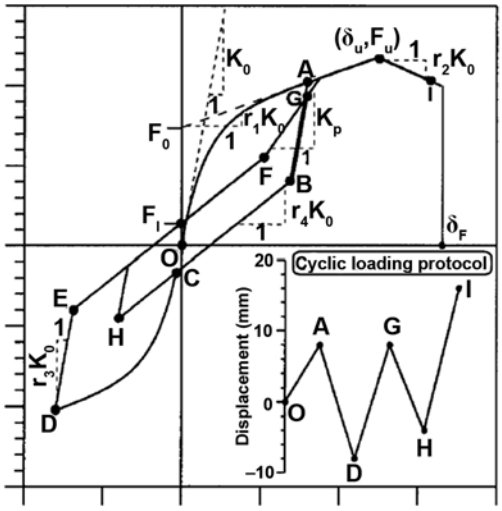


Figure 6.2. Loading paths and parameters.

as part of a CUREE-Caltech project (Caltech, California Institute of Technology, Pasadena, California, USA), was used in this study to characterize CLT shear wall behavior. The model requires 10 parameters to define force, stiffness, and their degradation as part of the hysteretic behavior of the shear wall. A generic 10-parameter hysteretic model is shown in Figure 6.2. Overturning restraining is not explicitly modeled. The 10-parameter model is fitted to shear wall data that incorporate various boundary conditions, tension rods for overturning, and shear connectors in accordance with the design method. The effects of a properly designed overturning restraint on shear wall response is addressed through fitting the hysteretic model to shear wall testing that incorporated the overturning restraint during the test.

The 10-parameter hysteretic model used as part of this project was calibrated using the test results. To account for the difference in panel height between the model (10 ft (3.1 m)) and the tests (8 ft (2.4 m)) while preserving the aspect ratio, the load-displacement test data were adjusted

Table 6.1—CLT wall test matrix (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m)

Test No.	Grade	Height (ft)	Length (ft)	No. of plys	Thickness (in.)	No. of connectors	Connector type	Gravity load (kip/ft)	Hold-down rod (No. and diam. in in.)
3	V2	8	4	5	6.65	3	A3	0.68	(2) 5/8
4	V2	8	4	5	6.65	3	A3	1.28	(2) 5/8
5	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
6[a]	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
9	V2	8	4	5	6.65	3	A3	—	(2) 5/8
10	V2	8	4	3	3.9	4	A3	—	(2) 5/8
11	V2	8	4	5	6.65	2	A3	—	(1) 5/8
13	E1	8	4	5	6.89	2	A3	—	(1) 5/8
14	E1	8	4	5	6.89	3	A3	—	(2) 5/8
15	E1	8	4	5	6.89	2	A3	—	(1) 5/8
17	E1	8	4	5	6.89	4	A3	—	(2) 5/8
18	V2	8	4	3	3.9	2	A3	—	(1) 5/8
19	V2	8	4	3	3.9	5	A3	—	(2) 3/4
20	V2	8	4	7	9.41	5	A3	—	(2) 3/4
21	V2	8	2	3	3.9	2	A3	—	(2) 5/8
23	V2	8	2 (2)	5	6.65	4	A3	—	(2) 5/8
24	E1	8	4	5	6.89	2	B3	—	(2) 5/8
25	E1	8	4	5	6.65	3	B3	—	(2) 3/4
26	V2	8	4 (2)	5	6.65	8	A3	—	(2) 5/8

[a]Test 6 was performed with the imposed boundary condition. The imposed boundary condition is explained in detail in Section 4.2.3.3.

by multiplying the force and displacement data from the tests by 1.25 (=10/8). The hysteretic model was then fitted to scaled load-displacement data. With this adjustment, using a panel length of 5 ft (1.5 m) in designs preserves the 2:1 aspect ratio and using a panel length of 2.5 ft (0.8 m) in designs preserves the 4:1 aspect ratio for an assumed 10-ft (3.1-m) wall height. The fitting was performed using the curve fitting tool in SAPWood, and the parameters were determined considering the average of the positive and negative envelope curves. The CLT test matrix is provided in Table 6.1, and the CUREE model parameters fit to the scaled test data are shown in Figures 6.3 to 6.9 and provided in Table 6.2.

Parameters are defined as follows (Pei and van de Lindt 2007):

K_0 is initial stiffness,

F_0 is resistance force parameter of the backbone,

F_1 is pinching residual resistance force,

r_1 is stiffness ratio parameter of the backbone, typically a small positive value,

r_2 is ratio of the degrading backbone stiffness to K_0, typically a negative value,

r_3 is ratio of the unloading path stiffness to K_0, typically close to 1,

r_4 is ratio of the pinching load path stiffness to K_0, typically under 0.1,

Δ_u is drift corresponding to the maximum restoring force of the backbone curve,

α is stiffness degradation parameter, usually takes a value between 0.5 and 0.9, and

β is strength degradation parameter, usually takes a value between 1.01 and 1.5.

Because wall lengths vary in a building, results from Test 21 (one panel), Test 23 (two panels), and Test 26 (four panels) were used to scale the parameters for varying wall lengths in the case of multipanel configurations comprised of panels with 4:1 aspect ratio. This approach is similar to the light-frame wood approach in which for a certain configuration, K_0, F_0, and F_1 parameters are given per wall length while the other parameters remain unchanged (Koliou and others 2018). In the case of CLT, one slight modification is with respect to Δ_u, where the values decreased based on the number of panels tested. For example, looking at Table 6.2, these are 8.75, 6.5, and 5.75 in. for one, two, and four multipanel configurations, respectively. However, this trend

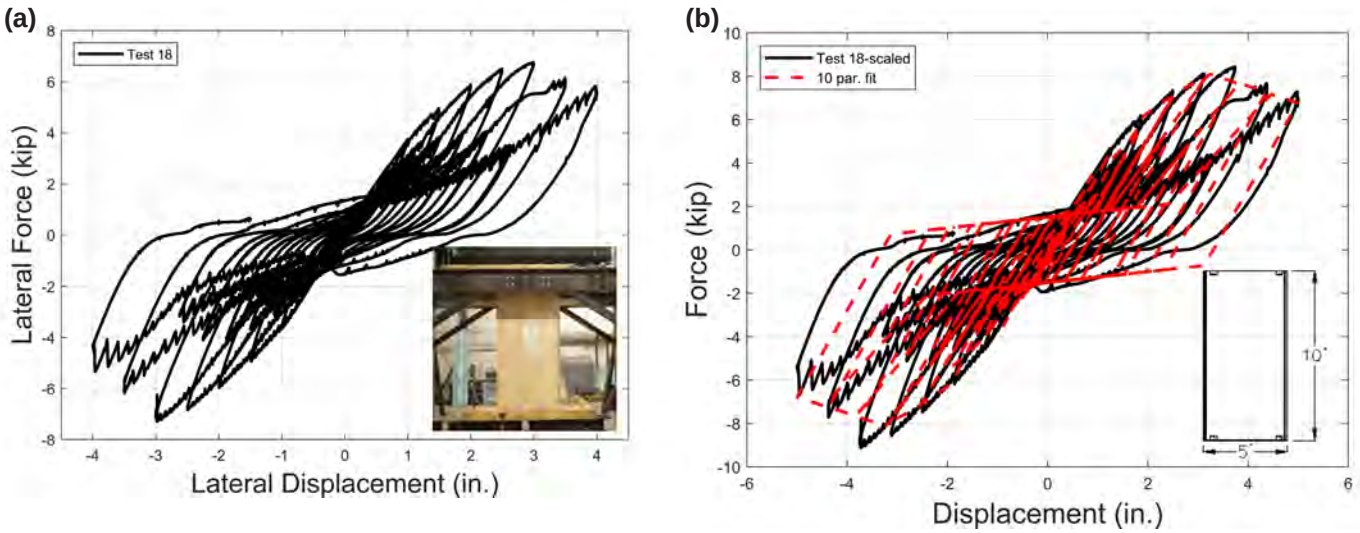


Figure 6.3. 2:1 aspect ratio panel, two connectors (a) Test 18, 4-ft by 8-ft by 3.9-in. (1.2-m by 2.4-m by 99-mm) panels; (b) Test 18, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

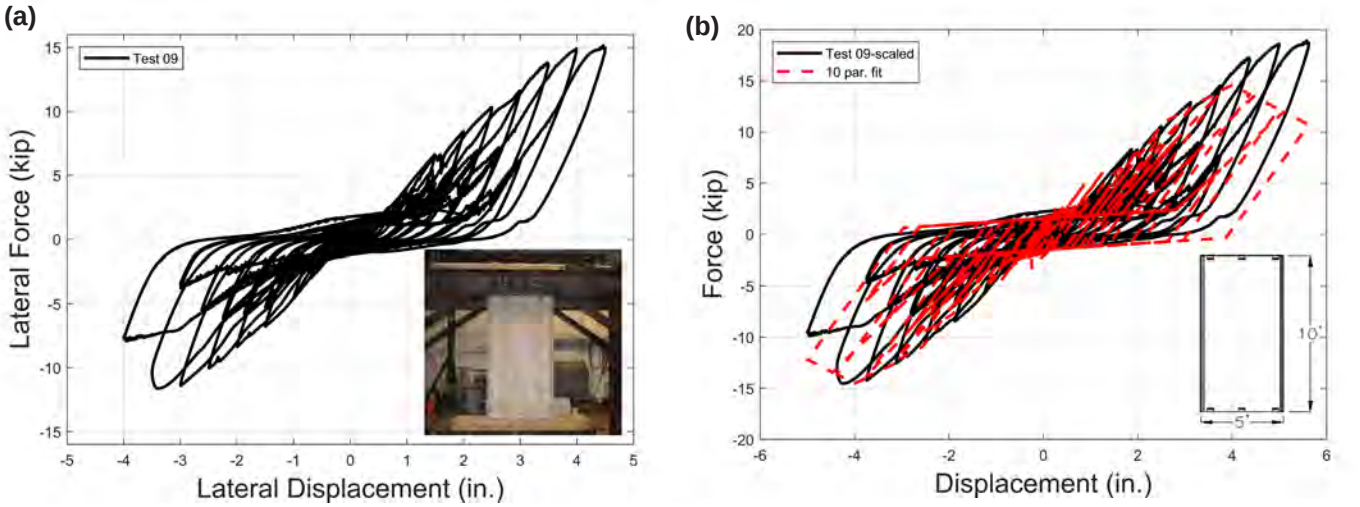


Figure 6.4. 2:1 aspect ratio panel, three connectors: (a) Test 09, 4-ft by 8-ft by 6.65-in. (1.2-m by 2.4-m by 165-mm) panels; (b) Test 09, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

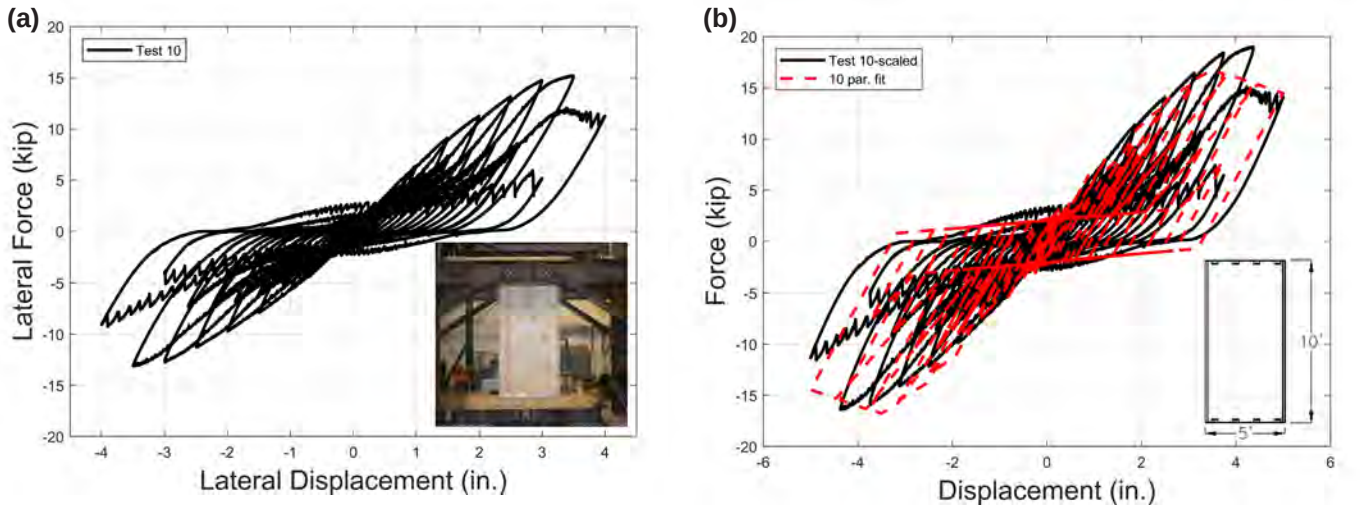


Figure 6.5. 2:1 aspect ratio panel, four connectors: (a) Test 10, 4-ft by 8-ft by 3.9-in. (1.2- m by 2.4-m by 99-mm) panels; (b) Test 10, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

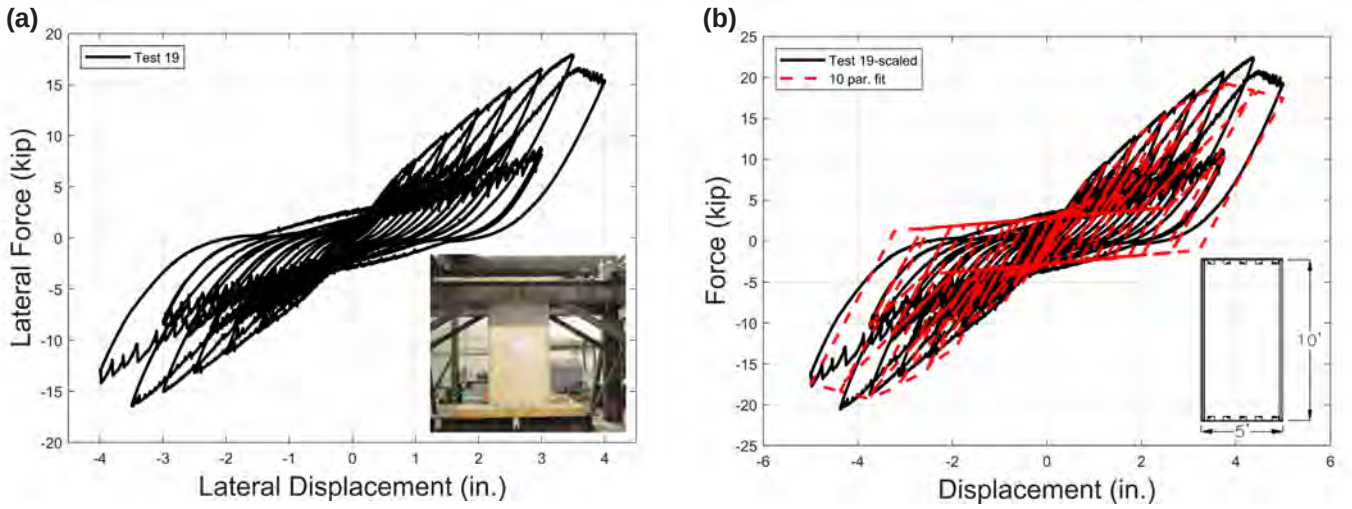


Figure 6.6. 2:1 aspect ratio panel, five connectors: (a) Test 19, 4-ft by 8-ft by 3.9-in. (1.2-m by 2.4-m by 99-mm) panels; (b) Test 19, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

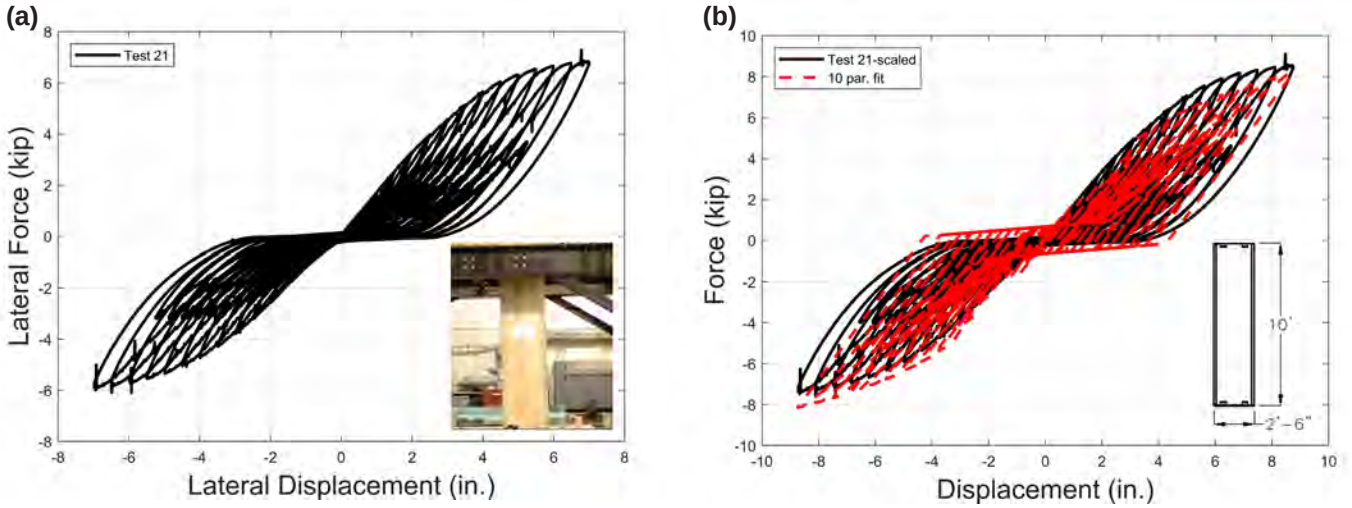


Figure 6.7. One 4:1 aspect ratio panel, two connectors per panel: (a) Test 21, one 2-ft by 8-ft by 3.9-in. (0.6-m by 2.4-m by 99-mm) panel; (b) Test 21, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

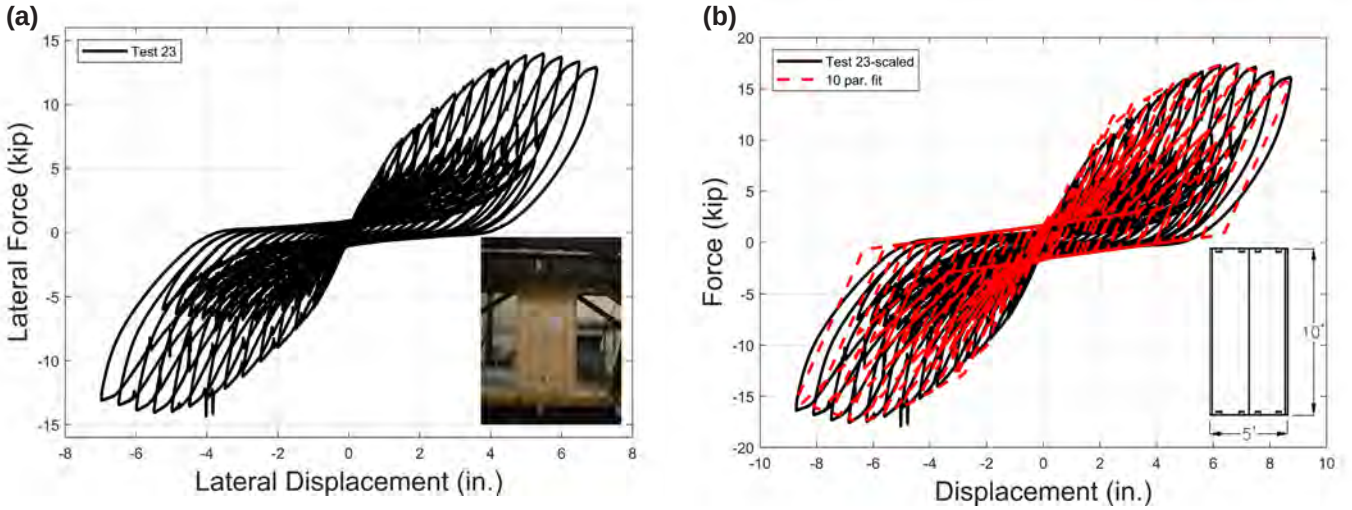


Figure 6.8. Two 4:1 aspect ratio panels, two connectors per panel: (a) Test 23, two 2-ft by 8-ft by 6.65-in. (0.6-m by 2.4-m by 165-mm) panels; (b) Test 23, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

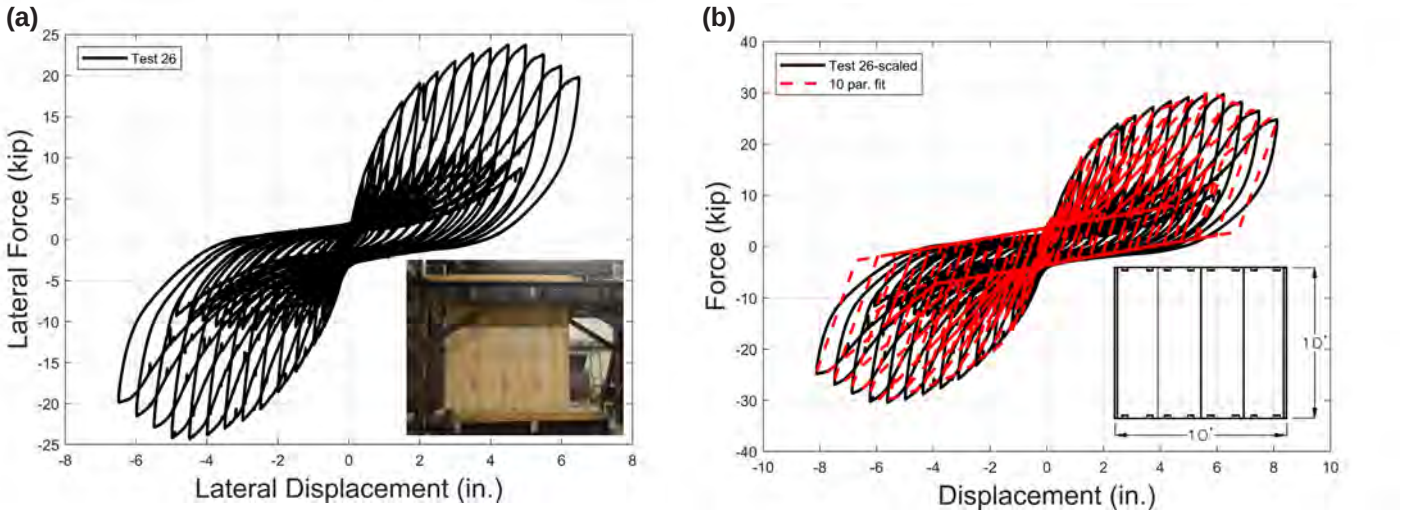


Figure 6.9. Four 4:1 aspect ratio panels, two connectors per panel: (a) Test 26, four 2-ft by 8-ft by 6.65-in. (0.6-m by 2.4-m by 165-mm) panels; (b) Test 26, scaled data and hysteretic fit (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Table 6.2—CUREE-SAWS[a] parameters used for analysis (1 ft = 0.3 m)

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	1	A3	2	S	5,000	8,760	1,500	0.05	–0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	S	6,000	16,700	1,500	0.1	–0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	S	8,000	17,670	2,000	0.125	–0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	S	10,000	21,460	2,750	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
2.5	1	A3	2	S	2,250	7,250	650	0.075	–0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	2	S	7,500	14,500	1,600	0.075	–0.125	0.825	0.05	6.5	0.75	1.05
2.5	4	A3	2	S	19,000	22,000	3,500	0.075	–0.125	0.825	0.05	5.75	0.75	1.05

[a]CUREE-SAWS, Consortium of Universities for Research in Earthquake Engineering-Seismic Analysis of Woodframe Structures.

was not a linear trend, and based on the rocking behavior of individual CLT wall panels, Δ_u for longer walls (greater than four panels) was taken as 5.75 in. For example, given K_0, F_0, and F_1 parameters for multipanel configurations in Table 6.2, parameters for the multipanel configuration comprised of five 4:1 aspect ratio panels are calculated as shown in Figure 6.10.

Parameters for the multipanel configuration with 4:1 aspect ratio panels along with 2:1 aspect ratio single-panel configuration tests were used to determine the hysteretic parameters for multipanel configurations comprised of 2:1 aspect ratio panels. Parameters for all different configurations are provided in Appendix G.

After the designs were performed for all the archetypes, SAPWood was used to assemble system level structural models to carry out the nonlinear time history analysis. The simplified model was shown previously in Figure 6.1.

6.2 Dynamic Analysis

IDA (Vamvatsikos and Cornell 2002) was performed using a suite of 22 bi-axial ground motions identified in FEMA P695 as far-field ground motions. The engineering demand parameter (EDP) was interstory drift, and intensity measure (IM) was spectral acceleration. The ground motions were scaled to increasing intensities to determine median collapse intensity, S_{CT}, which is defined as the intensity at which 22 of the 44 ground motions cause collapse. Although a full IDA analysis is not necessary to determine median collapse intensity (S_{CT}), for the purpose of this study, ground motions were scaled to increasing intensities that ranged from 1 to 5 *g* and the IDA results were then used to develop collapse fragility.

The scaling was performed in accordance with FEMA P695 methodology, in which a record set is scaled by a single factor such that the median response spectrum of the set only matches the spectral acceleration of interest at the fundamental period of the building. Figure 6.11 shows an example of these ground motions, scaled to MCE demand.

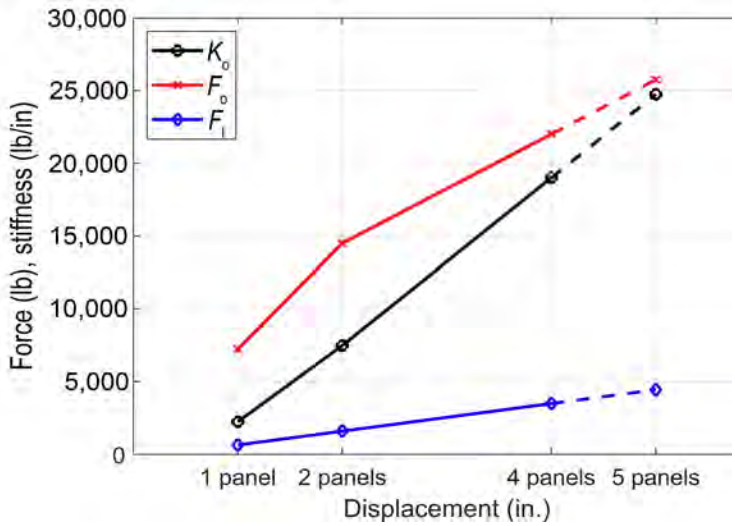


Figure 6.10. 4:1 aspect ratio multipanel configuration parameter scaling (1 lb = 4.45 N; 1 lb/in = 175 N/m).

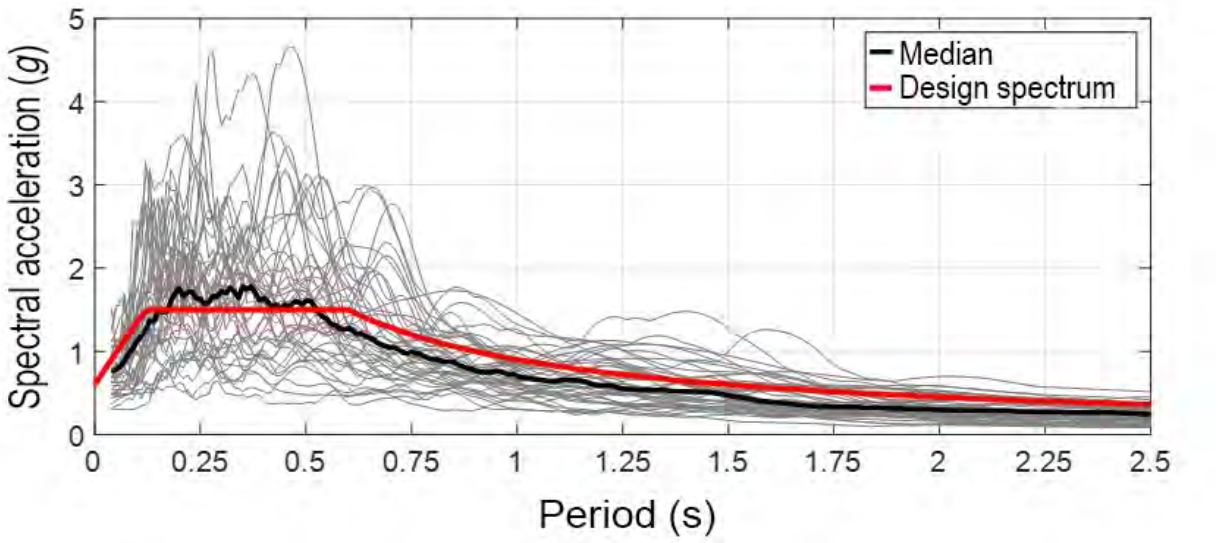


Figure 6.11. FEMA P695 far-field ground motions scaled to maximum considered earthquake demand at T = 0.265 s (1 g = 9.8 m/s^2).

As previously mentioned, the fundamental period T was calculated based on the following equation (T is not permitted to exceed):

$$T = C_u T_a = C_u C_t h_n^x \geq 0.25 \text{ s}$$

h_n is height of the structure,

C_t = 0.02 (ASCE 2016, table 12.8-2),

C_u = 1.40 (ASCE 2016, table 12.8-1), and

x = 0.75 (ASCE 2016, table 12.8-2).

The upper limit was used because, analytically, T was found to be larger.

6.3 Damping

Dynamic response of a structure is significantly influenced by its damping properties. In a structure subjected to dynamic loads, energy is dissipated through hysteretic damping and viscous damping. For modeling purposes, hysteretic damping is directly considered in hysteretic modeling of the CLT shear walls whereas viscous damping is considered in the form of Rayleigh damping as a percentage of critical damping.

Based on a comprehensive study of light-frame wood structures (Beck and others 2002), the measured values of viscous damping ranged from 2.6% to 17.3% of the critical damping. The study included three distinct methods: (a) measurements of the structural response under seismic events, (b) forced vibration response, and (c) shake table

response of a two-story structure. The seismic measurement performed for five buildings, which ranged from one to three stories, indicated damping ratios ranging from 6.3% to 17.3%. The damping ratios obtained from force vibration tests on two two-story structures and one three-story structure ranged from 2.6% to 6.8%. For the two-story structures (Folz and Filiatrault 2004), the average damping ratio was close to 7%.

In the case of the special CLT shear wall system, perpendicular walls, nonstructural components, and CLT walls in addition to the shear walls in a wall line contributed to the damping. According to the FEMA P695 methodology, damping can be in the range of 2% to 5% of critical damping, and for the purpose of this study, the conservative lower bound of 2% critical damping was assumed. It is important to point out that previous studies of SAPWood, cited earlier in Section 6.1, have shown appropriate consideration of the energy dissipation associated with damping.

6.4 Static Pushover Analysis

Static pushover analysis was performed for each archetype to determine maximum base shear resistance, V_{max}, and ultimate displacement, δ_u, which were in turn used to determine overstrength factor, Ω, and period-based ductility, μ_T.

Archetypes were subjected to vertically distributed load patterns proportional to their corresponding fundamental mode as the lateral displacement was increased monotonically. The numerical model for this purpose was developed in OpenSees (Regents of the University of California, Oakland, California, USA) (McKenna and others 2011). The developed model is similar to the SAPWood model in which diaphragms are assumed to be rigid and CLT shear walls are presented using the phenomenological CUREE model. The model accounts for degradation in stiffness and strength, as well as pinching behavior.

Second order (P-Δ) effects were considered in the pushover analysis using leaning columns (Geschwindner 2002). At each floor, the diaphragm was connected to the leaning column to transfer the P-Δ. The base shear in this case was the sum of all the shear walls in the first floor and shear at the base of the leaning column.

Chapter 7
Performance Evaluation

Collapse criteria were used with IDA results to develop collapse fragility function, which is the failure probability of the proposed SFRS with respect to spectral acceleration. The seismic performance factors were evaluated and the adjusted collapse margin ratio (ACMR) calculated. If the ACMR meets the collapse criteria, the trial *R* value and other calculated seismic performance factors are considered acceptable per the methodology. However, if not, various steps of the P695 methodology are repeated to obtain the desired ACMR.

7.1 Collapse Criteria

To evaluate collapse of CLT special shear wall system, a limit was placed on interstory drift for nonsimulated collapse mode. Based on the CLT shear wall test results, shown in Tables 7.1 and 7.2, interstory drift ratios of 4.5% and 5.5% were assumed for low aspect ratio and high aspect ratio cases, respectively. If low and high aspect ratios are mixed within an archetype wall line, the interstory drift ratio of 4.5% was assumed to be associated with collapse.

7.2 Total System Collapse Uncertainty

In addition to the systematic approach consisting of nonlinear static and dynamic analyses, the FEMA P695 methodology takes into account uncertainties inherent in the design requirements (DR), test data (TD), modeling methods (MDL), and variation in the ground motion records (RTR). Record-to-record uncertainty considers variability in the response of the index archetypes caused by ground motions. Design requirements uncertainty and test data uncertainty account for completeness and robustness of the design requirements and the test data, respectively. Modeling uncertainty is related to the accuracy of the models to capture collapse performance and the extent of archetypes to represent a range of structural performance.

Total collapse uncertainty is given by the following equation:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \qquad (7.1)$$

where β_{TOT} is the total system collapse uncertainty,

β_{RTR} is the record-to-record collapse uncertainty,

β_{DR} is the design requirement collapse uncertainty,

β_{TD} is the test-data-related collapse uncertainty, and

β_{MDL} modeling-related collapse uncertainty.

β_{RTR} was calculated for each archetype using the following equation:

$$\beta_{RTR} = 0.1 + \mu_T \leq 0.40 \qquad (7.2)$$

Collapse evaluation requires calculation of total uncertainty using Equation 7.1. The quality ratings associated with the ground motion records, design requirements, test data, and modeling for the purpose of this study are shown in Tables 7.3 to 7.6. The approach here was similar to the light-frame wood structure example. The quality ratings were selected considering the test data, the design requirements that are based on historic design provisions of NDS, and the modeling approach. These ratings were assigned based on a review of FEMA P695 descriptions of quality ratings and comparison with the ratings assigned to light-frame wood shear wall examples in FEMA P695. Although special aspects of design include use of prescribed connectors and special design criteria for overturning to ensure development of connector shear capacity, a rating of "good" was given as opposed to the "superior" used for light-frame wood shear walls because the CLT shear wall is a new system. Modeling employed the same techniques as used for light-frame wood except that nonsimulated collapse drift limits of 4.5% and 5.5% were imposed based on CLT panel aspect ratios, which is conservative relative to larger drifts observed in the tests. Test data were also regarded as "good" because they involved both connector level testing and fully reversed cyclic shear wall testing under different boundary conditions for the prescribed connectors permitted by the design requirements. Tested components that formed the CLT shear wall were also addressed by existing code reference design standards.

7.3 Collapse Margin Ratio and Adjusted Collapse Margin Ratio

The collapse margin ratio (CMR) for each archetype was calculated using the following equation:

$$CMR_i = \frac{\hat{S}_{CT_i}}{S_{MT}} \qquad (7.3)$$

where $\hat{S}_{CT}$ is the median collapse intensity and S_{MT} is the MCE ground motion intensity.

CMR was then adjusted (ACMR) using the spectral shape factor (SSF) to account for the effects of spectral shape:

$$ACMR = CMR \times SSF \qquad (7.4)$$

SSF was determined based on the SDC for which the archetype was designed, the code-based period, and the period-based ductility obtained from the pushover

analysis. For the archetypes presented in this report, SSF was determined using the data provided in Table 7.7. As previously presented in Equation (3.5), the code-based period was calculated as follows:

$$T = C_u T_a = C_u C_t h_n^x = 0.02 \times 1.4 \times h_n^{0.75} = 0.028 h_n^{0.75}$$

where h_n is height of the structure.

The period-based ductility was obtained from the pushover analyses using the following equation (a sample pushover curve is shown in Fig. 7.1):

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{7.5}$$

where δ_u is the roof displacement corresponding to 80% post peak load ($0.8V_{max}$) and $\delta_{y,eff}$ is the effective yield roof displacement.

7.4 Overstrength Factor, Ω_0

Overstrength, Ω, for an individual archetype was calculated using the following equation, and the average value was calculated for each performance group:

$$\Omega = \frac{V_{max}}{V} \tag{7.6}$$

Based on FEMA P695, the system overstrength factor, Ω_0, should be larger than the largest value of Ω calculated for the performance groups. Additionally, Ω_0 is not to exceed 1.5*R* and should be less than the upper limit of 3.0, which is imposed for practical reasons and for consistency with the largest value of Ω_0 provided in table 12.2-1 of ASCE 7-16 for the current systems.

7.5 Deflection Amplification Factor, C_d

The deflection amplification factor, C_d, is calculated using the following equation:

$$C_d = \frac{R}{\beta_I} \tag{7.7}$$

As indicated in FEMA P695, inherent damping (β_I) may be assumed to be 5% of critical damping, which gives a corresponding value of β_I = 1.0 based on table 18.7-1 of ASCE 7-16. This results in $C_d = R$.

All the archetypes were evaluated for R = 3. Only archetypes with high aspect ratio panel only configurations were checked for R = 4. This was because of the better deformation capacity demonstrated by the high aspect ratio panel only configurations during the shear wall testing. All archetype designs are provided in Appendix B. An example application is presented in Appendix H. The example is intended to demonstrate extraction of an archetype from an index building, its seismic design using the design methodology developed as part of this project, nonlinear static and dynamic analyses, and evaluation of the results.

Table 7.1—Top of wall maximum displacement observed in CLT shear wall tests (1 ft = 0.3 m; 1 in. = 25.4 mm)

Test No.	Length (ft)	No. of connectors	Connector type	Δ_{max} (in.)			Average drift (% wall height)
				+ve	–ve	Avg.	
3	4	3	A3	4.49	4.48	4.5	4.67
4	4	3	A3	4.5	4.49	4.5	4.68
5	4	3	A3	6	6	6.0	6.25
6*	4	3	A3	6	6	6.0	6.25
9	4	3	A3	4.5	4	4.3	4.43
10	4	4	A3	4	4	4.0	4.17
11	4	2	A3	4.47	4.5	4.5	4.67
13	4	2	A3	5.5	5	5.3	5.47
14	4	3	A3	5.5	5.5	5.5	5.73
15	4	2	A3	5	4.5	4.8	4.95
17	4	4	A3	5	4.5	4.8	4.95
18	4	2	A3	4	4	4.0	4.17
19	4	5	A3	4	4	4.0	4.17
20	4	5	A3	4	4	4.0	4.17
21	2	2	A3	7	7	7.0	7.29
23	2 (2)	4	A3	7	7	7.0	7.29
26	4 (2)	8	A3	6.5	6.5	6.5	6.77

Table 7.2—Drift data summary

Panel aspect ratio	Connector type	Avg. max. drift (% wall height)
4:1	A3	7.12
2:1	A3	4.91

Table 7.3—Quality rating of design requirements (after table 3-1 in FEMA P695)

Completeness and robustness	Confidence in basis of design requirements		
	High	Medium	Low
High: Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	Superior β_{DR} = 0.10	Good β_{DR} = 0.20	Fair β_{DR} = 0.35
Medium: Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	Good β_{DR} = 0.20	Fair β_{DR} = 0.35	Poor β_{DR} = 0.50
Low: Questionable safeguards against unanticipated failure modes. Many important design and quality assurance issues are not addressed.	Fair β_{DR} = 0.35	Poor β_{DR} = 0.50	—

Table 7.4—Quality rating of test data (after table 3-2 in FEMA P695)

Completeness and robustness	Confidence in basis of design requirements		
	High	Medium	Low
High: Material, component, connection, assembly, and system behavior well understood and accounted for. All, or nearly all, important testing issues addressed.	Superior $\beta_{TD} = 0.10$	Good $\beta_{TD} = 0.20$	Fair $\beta_{TD} = 0.35$
Medium: Material, component, connection, assembly, and system behavior generally understood and accounted for. Most important testing issues addressed.	Good $\beta_{TD} = 0.20$	Fair $\beta_{TD} = 0.35$	Poor $\beta_{TD} = 0.50$
Low: Material, component, connection, assembly, and system behavior fairly understood and accounted for. Several important testing issues not addressed.	Fair $\beta_{TD} = 0.35$	Poor $\beta_{TD} = 0.50$	—

Table 7.5—Quality rating of index archetype models (after table 5-3 in FEMA P695)

Representation of collapse characteristics	Accuracy and robustness of models		
	High	Medium	Low
High: Index models capture the full range of the archetype design space and structural behavioral effects that contribute to collapse.	Superior $\beta_{MDL} = 0.10$	Good $\beta_{MDL} = 0.20$	Fair $\beta_{MDL} = 0.35$
Medium: Index models are generally comprehensive and representative of the archetype design space and structural behavioral effects that contribute to collapse.	Good $\beta_{MDL} = 0.20$	Fair $\beta_{MDL} = 0.35$	Poor $\beta_{MDL} = 0.50$
Low: Significant aspects of the design space and/or collapse behavior are not captured in the index archetypes.	Fair $\beta_{MDL} = 0.35$	Poor $\beta_{MDL} = 0.50$	—

Table 7.6—Quality ratings used for evaluation

Uncertainty	Quality rating value	Description
Record-to-record (β_{RTR})	0.40	Equation 7.2
Design requirements (β_{DR})	0.20	Good: Medium in completeness and robustness and high confidence
Test data (β_{TD})	0.20	Good: Medium in completeness and robustness and high confidence
Modeling (β_{MDL})	0.20	Good: Medium in completeness and robustness and high confidence

Table 7.7—Spectral shape factor for archetypes designed using seismic design category D_{max} (after table 7-1b in FEMA P695)

	Period-based ductility, μ_T							
T (s)	1.0	1.1	1.5	2.0	3.0	4.0	6.0	≥8.0
≤0.5	1.00	1.05	1.10	1.13	1.18	1.22	1.28	1.33
0.6	1.00	1.05	1.11	1.14	1.20	1.24	1.30	1.36
0.7	1.00	1.06	1.11	1.15	1.21	1.25	1.32	1.38
0.8	1.00	1.06	1.12	1.16	1.22	1.27	1.35	1.41
0.9	1.00	1.06	1.13	1.17	1.24	1.29	1.37	1.44
1.0	1.00	1.07	1.13	1.18	1.25	1.31	1.39	1.46
1.1	1.00	1.07	1.14	1.19	1.27	1.32	1.41	1.49
1.2	1.00	1.07	1.15	1.20	1.28	1.34	1.44	1.52
1.3	1.00	1.08	1.16	1.21	1.29	1.36	1.46	1.55
1.4	1.00	1.08	1.16	1.22	1.31	1.38	1.49	1.58
≥1.5	1.00	1.08	1.17	1.23	1.32	1.40	1.51	1.61

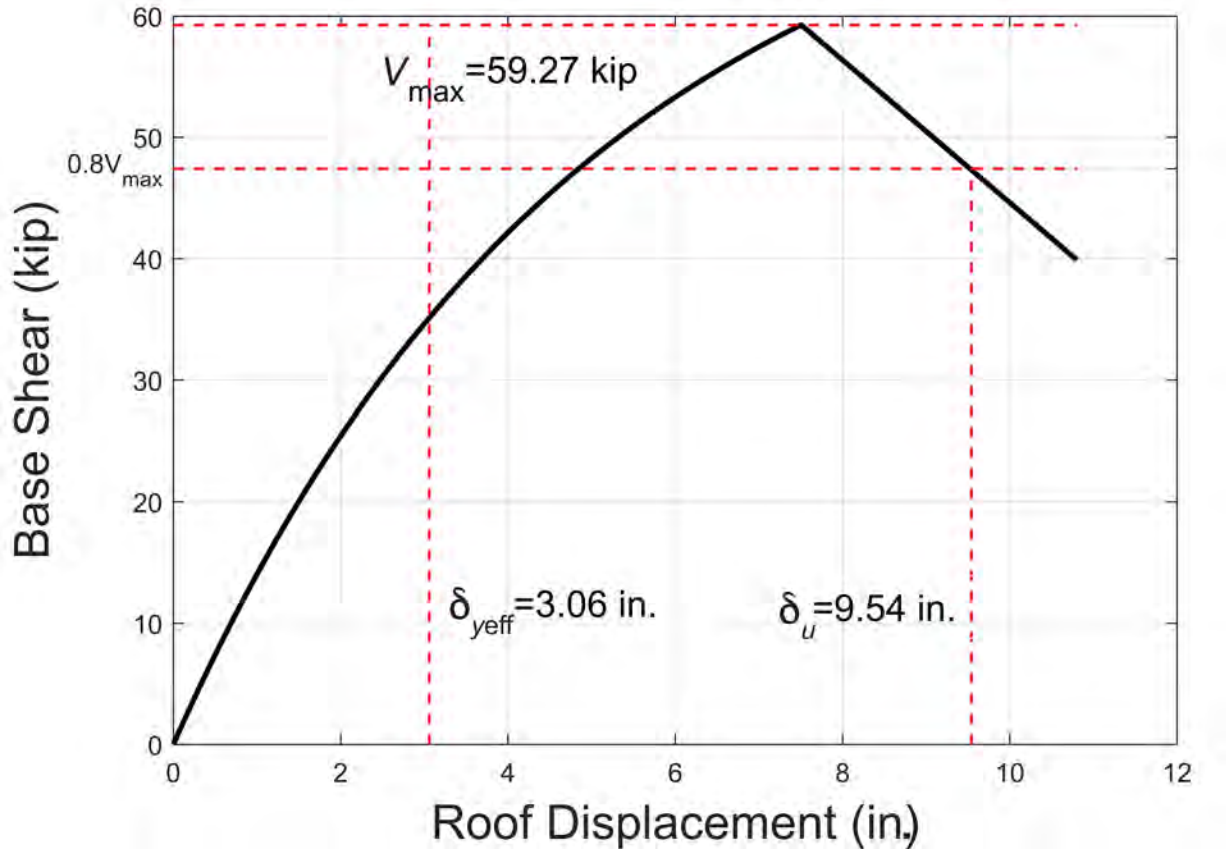


Figure 7.1. Sample nonlinear static pushover analysis (1 kip = 1,000 lb = 0.45 metric ton; 1 in. = 25.4 mm).

Chapter 8
Results

8.1 Response Modification Factor, *R*, for CLT Shear Walls

As mentioned previously, the procedure presented in Appendix H was carried out for $R = 3$ for all the archetypes and $R = 4$ for archetypes with high aspect ratio panel configurations only. Seismic design properties and final results and acceptance criteria are provided in Tables 8.1 and 8.2 for the $R = 3$ case and Tables 8.3 and 8.4 for the $R = 4$ case, respectively. Pushover curves and collapse fragility curves for all the archetypes are shown in Appendix I. The red lines in the static pushover curves represent $0.8V_{max}$ for the corresponding curves. Deformation shapes for some of the archetypes are provided in Appendix J.

Based on the results shown in Table 8.2, all the individual archetypes designed for $R = 3$ passed the ACMR20% criteria and the average of the performance groups exceeded the ACMR10% criteria for the performance groups. Similarly, looking at the results shown in Tables 8.4 for $R = 4$ archetypes with high aspect ratio panel configurations, all the individual archetypes and performance groups passed their corresponding criteria. Summaries of the ACMR for the performance groups are provided in Figures 8.1 and 8.2 for the $R = 3$ and $R = 4$ cases, respectively.

8.2 Overstrength Factor, Ω_0, CLT Special Shear Walls

For the $R = 3$ case for all the archetypes, the overstrength factor calculated for the archetypes ranged from 1.8 to 4.85 with most values centered around 3 and the average overstrength factors for the performance groups ranged from 2.29 to 3.53. Similarly, for the $R = 4$ case for archetypes with high aspect ratio panel configurations, the overstrength factors ranged from 2.34 to 5.25 with most values centered around 3. The average of the performance groups ranged from 2.02 to 4.03. Because the largest average of the performance groups was greater than 3, the upper limit of $\Omega_0 = 3.0$ was considered for the system for both cases.

8.3 Deflection Amplification Factor for CLT Special Shear Walls

As previously explained in Section 7.5, C_d is equal to R when β_I is set as 1.0. Therefore, for the $R = 3$ case, $C_d = 3.0$. In the case of $R = 4$, archetypes were designed using $C_d = 3$ but $C_d = 4$ was proposed for purposes of design consistent with the FEMA P695 recommendation for $C_d = R$. For the $R = 4$ system, use of $C_d = 3$ in the archetype design and analysis produced equal or smaller ACMRs than would be associated with $C_d = 4$. ACMRs associated with archetype designs based on $C_d = 4$ will be the same or increased relative to $C_d = 3$ because of the potential for archetype designs to require added connectors to increase stiffness (and strength) to meet seismic drift limits.

Table 8.1—Archetype seismic design properties, *R* = 3 (1 kip = 1,000 lb = 0.45 metric ton)[a]

		Key archetype design parameters							
			Seismic design criteria						
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			PG-01						
1	1_E_3_1_LR_HG_DX_SP	Low	High	D_{max}	0.359	0.59	17.43	53.90	1.50
2	2_4_2_1_LR_HG_DX_SP	Low	High	D_{max}	0.265	0.53	19.30	67.20	1.50
3	6_D_2_1_LR_HG_DX_SP	Low	High	D_{max}	0.265	0.47	8.20	24.60	1.50
			PG-02						
4	4_3_ 6_1_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.77	34.79	105.00	1.49
5	8_3_6_1_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.54	30.31	90.50	1.49
6	9_B_6 _1_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.60	34.64	104.60	1.49
			PG-05						
13	1_E_3_1_LR_LG_DX_SP	Low	Low	D_{max}	0.359	0.61	14.83	46.02	1.50
14	2_4_2_1_LR_LG_DX_SP	Low	Low	D_{max}	0.265	0.53	17.33	61.34	1.50
15	6_D_2_1_LR_LG_DX_SP	Low	Low	D_{max}	0.265	0.49	7.14	21.40	1.50
			PG-06						
16	4_3_ 6_1_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.78	28.91	87.24	1.49
17	8_3_6_1_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.54	24.67	74.45	1.49
18	9_B_6 _1_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.60	27.95	84.35	1.49
			PG-09						
25	1_E_3_1_HR_HG_DX_SP	High	High	D_{max}	0.359	0.53	17.43	53.90	1.50
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	0.265	0.47	19.30	67.20	1.50
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	0.265	0.48	8.20	24.60	1.50
			PG-10						
28	4_3_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.74	34.79	105.00	1.49
29	8_3_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.66	30.31	90.50	1.49
30	9_B_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.67	34.64	104.60	1.49

Table 8.1—Archetype seismic design properties, *R* = 3 (1 kip = 1,000 lb = 0.45 metric ton)[a] (con.)

Archetype No.	Archetype ID	Key archetype design parameters: Panel aspect ratio	Seismic design criteria: Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			PG-13						
37	1_E_3_1_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.55	14.83	46.02	1.50
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.46	17.33	61.38	1.50
39	6_D_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.47	7.10	21.40	1.50
			PG-14						
40	4_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.73	28.91	87.24	1.49
41	8_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.64	24.67	74.75	1.49
42	9_B_6_1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.68	27.95	84.35	1.49
			PG-17						
49	8_3_4_1_MR_HG_DX_SP	Mix	High	D_{max}	0.45	0.62	38.92	115.80	1.50
50	4_3_4_1_MR_HG_DX_LP	Mix	High	D_{max}	0.45	0.63	44.80	134.43	1.50
51	7_3_3_1_MR_HG_DX_SP	Mix	High	D_{max}	0.36	0.39	53.98	161.90	1.50
			PG-18						
52	4_3_6_1_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.72	34.79	105.00	1.49
53	8_3_6_1_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.58	30.31	90.50	1.49
54	4_B_6_1_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.74	31.04	93.68	1.49
			PG-21						
61	8_3_4_1_MR_LG_DX_SP	Mix	Low	D_{max}	0.445	0.64	32.17	96.52	1.50
62	4_3_4_1_MR_LG_DX_SP	Mix	Low	D_{max}	0.445	0.62	37.71	113.12	1.50
63	7_3_3_1_MR_LG_DX_SP	Mix	Low	D_{max}	0.359	0.38	45.52	136.55	1.50
			PG-22						
64	4_3_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.75	28.91	87.24	1.49
65	8_3_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.71	24.70	74.45	1.49
66	4_B_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.74	25.79	77.83	1.49

Table 8.1—Archetype seismic design properties, *R* = 3 (1 kip = 1,000 lb = 0.45 metric ton)[a] (con.)

		Key archetype design parameters							
			Seismic design criteria						
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			PG-25						
73	2_2_2_2_LR_HG_DX_SP	Low	High	D_{max}	0.265	0.48	24.16	71.70	1.50
74	3_2_1_2_LR_HG_DX_SP	Low	High	D_{max}	0.250	0.29	14.73	44.20	1.50
75	5_B_2_2_LR_HG_DX_SP	Low	High	D_{max}	0.265	0.47	62.73	188.20	1.50
			PG-26						
76	4_E_6_2_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.77	54.77	165.30	1.49
77	8_B_6_2_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.55	61.70	184.30	1.49
78	9_3_6_2_LR_HG_DX_LP	Low	High	D_{max}	0.604	0.50	98.56	297.60	1.49
			PG-29						
85	2_2_2_2_LR_LG_DX_SP	Low	Low	D_{max}	0.265	0.46	21.71	64.32	1.50
86[c]	3_2_1_2_LR_LG_DX_SP	Low	Low	D_{max}	0.250	0.29	14.73	44.20	1.50
87	5_B_2_2_LR_LG_DX_SP	Low	Low	D_{max}	0.265	0.51	53.78	161.36	1.50
			PG-30						
88	4_E_6_2_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.77	45.51	137.40	1.49
89	8_B_6_2_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.54	50.22	151.57	1.49
90	9_3_6_2_LR_LG_DX_LP	Low	Low	D_{max}	0.604	0.53	79.51	239.98	1.49
			PG-33						
97	1_A_3_2_HR_HG_DX_SP	High	High	D_{max}	0.359	0.52	26.02	74.40	1.50
98	6_E_2_2_HR_HG_DX_SP	High	High	D_{max}	0.265	0.45	23.15	69.60	1.50
99	7_A_3_2_HR_HG_DX_SP	High	High	D_{max}	0.359	0.53	79.24	237.70	1.50
			PG-34						
100	4_E_6_2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.69	54.77	165.30	1.49
101	8_B_6_2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.64	61.70	184.30	1.49
102	9_3_6 _2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.63	98.56	297.60	1.49

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Table 8.1—Archetype seismic design properties, *R* = 3 (1 kip = 1,000 lb = 0.45 metric ton)[a] (con.)

Archetype No.	Archetype ID	Key archetype design parameters: Panel aspect ratio	Seismic design criteria: Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			PG-37						
109	1_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.49	22.15	63.36	1.50
110	6_E_2_2_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.50	20.19	60.57	1.50
111	7_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.52	66.82	200.40	1.50
			PG-38						
112	4_E_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.75	45.52	137.40	1.49
113	8_B_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.66	50.20	151.57	1.49
114	9_3_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.60	79.51	239.98	1.49
			PG-41						
121	2_2_2_2_MR_HG_DX_SP	Mix	High	D_{max}	0.265	0.46	24.16	71.70	1.50
122[d]	3_2_1_2_MR_HG_DX_SP	Mix	High	D_{max}	0.250	0.28	14.73	44.20	1.50
123	5_B_2_2_MR_HG_DX_SP	Mix	High	D_{max}	0.265	0.50	62.73	188.20	1.50
			PG-42						
124	4_E_6_2_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.74	54.77	165.30	1.49
125	8_B_6_2_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.60	61.70	184.30	1.49
126	9_3_6_2_MR_HG_DX_LP	Mix	High	D_{max}	0.604	0.56	98.56	297.60	1.49
			PG-45						
133	2_2_2_2_MR_LG_DX_SP	Mix	Low	D_{max}	0.265	0.46	21.71	64.32	1.50
134	3_2_1_2_MR_LG_DX_SP	Mix	Low	D_{max}	0.250	0.28	14.73	44.20	1.50
135	5_B_2_2_MR_LG_DX_SP	Mix	Low	D_{max}	0.265	0.51	53.78	161.36	1.50
			PG-46						
136	4_E_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.74	45.51	137.40	1.49
137	8_B_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.67	50.20	151.57	1.49
138	9_3_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	0.604	0.55	79.51	239.98	1.49

[a]This table presents six performance groups (PG) each including three archetypes.
[b]SDC, seismic design category.
[c]Archetype 86 is the same as archetype 74 because for the one-story archetype, LG and HG cases are the same.
[d]Archetype 134 is the same as archetype 122 because for the one-story archetype, LG and HG cases are the same.

Table 8.2—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 3

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
					Performance group (PG)-01							
1	1_E_3_1_LR_HG_DX_SP	Low	High	D_{max}	3.02	2.66	3.06	2.04	1.16	2.37	1.53	PASS
2	2_4_2_1_LR_HG_DX_SP	Low	High	D_{max}	3.07	3.12	2.54	1.70	1.18	2.01	1.56	PASS
3	6_D_2_1_LR_HG_DX_SP	Low	High	D_{max}	2.99	2.65	2.67	1.78	1.16	2.07	1.52	PASS
Mean of PG					3.03			1.84		2.15	1.93	PASS
					PG-02							
4	4_3_6_1_LR_HG_DX_LP	Low	High	D_{max}	3.03	2.38	3.22	2.16	1.16	2.51	1.50	PASS
5	8_3_6_1_LR_HG_DX_LP	Low	High	D_{max}	2.98	2.03	3.39	2.27	1.14	2.59	1.47	PASS
6	9_B_6_1_LR_HG_DX_LP	Low	High	D_{max}	3.34	2.75	3.53	2.37	1.19	2.81	1.54	PASS
Mean of PG					3.11			2.27		2.64	1.86	PASS
					PG-05							
13	1_E_3_1_LR_LG_DX_SP	Low	Low	D_{max}	3.07	2.34	3.08	2.05	1.15	2.35	1.50	PASS
14	2_4_2_1_LR_LG_DX_SP	Low	Low	D_{max}	3.02	2.65	2.53	1.69	1.16	1.96	1.52	PASS
15	6_D_2_1_LR_LG_DX_SP	Low	Low	D_{max}	3.17	2.51	2.81	1.88	1.16	2.17	1.51	PASS
Mean of PG					3.09			1.87		2.16	1.88	PASS
					PG-06							
16	1_E_3_1_HR_HG_DX_SP	Low	Low	D_{max}	3.06	2.50	3.15	2.11	1.17	2.47	1.51	PASS
17	2_4_2_1_HR_HG_DX_SP	Low	Low	D_{max}	3.29	2.78	3.70	2.48	1.19	2.94	1.54	PASS
18	6_D_2_1_HR_HG_DX_SP	Low	Low	D_{max}	3.47	2.57	3.51	2.35	1.17	2.76	1.52	PASS
Mean of PG					3.27			2.32		2.73	1.90	PASS
					PG-09							
25	1_E_3_1_HR_HG_DX_SP	High	High	D_{max}	2.74	4.37	3.27	2.18	1.23	2.68	1.56	PASS
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	3.56	4.45	3.06	2.04	1.23	2.51	1.56	PASS
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	3.09	3.98	2.96	1.97	1.22	2.40	1.56	PASS
Mean of PG					3.13			2.06		2.53	1.97	PASS
					PG-10							
28	4_3_6_1_HR_HG_DX_LP	High	High	D_{max}	3.09	3.40	4.19	2.81	1.22	3.41	1.56	PASS
29	8_3_6_1_HR_HG_DX_LP	High	High	D_{max}	2.67	5.73	3.92	2.63	1.29	3.40	1.56	PASS
30	9_B_6_1_HR_HG_DX_LP	High	High	D_{max}	2.94	4.39	4.24	2.84	1.25	3.56	1.56	PASS
Mean of PG					2.90			2.76		3.46	1.97	PASS

Table 8.2—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 3 (con.)

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
					PG-13							
37	1_E_3_1_HR_LG_DX_SP	High	Low	D_{max}	3.12	3.84	3.54	2.36	1.21	2.86	1.56	PASS
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	3.98	4.19	3.27	2.18	1.23	2.67	1.56	PASS
39	6_D_2_1_HR_LG_DX_SP	High	Low	D_{max}	3.48	3.61	2.90	1.93	1.20	2.33	1.56	PASS
Mean of PG					3.53			2.16		2.62	1.97	PASS
					PG-14							
40	4_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	3.07	3.28	3.91	2.62	1.21	3.18	1.56	PASS
41	8_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	2.93	5.45	4.47	3.00	1.28	3.85	1.56	PASS
42	9_B_6_1_HR_LG_DX_LP	High	Low	D_{max}	2.97	4.12	3.89	2.61	1.24	3.24	1.56	PASS
Mean of PG					2.99			2.74		3.42	1.97	PASS
					PG-17							
49	8_3_4_1_MR_HG_DX_SP	Mix	High	D_{max}	2.19	3.63	3.25	2.17	1.21	2.61	1.56	PASS
50	4_3_ 4_1_MR_HG_DX_LP	Mix	High	D_{max}	2.03	3.74	3.12	2.08	1.21	2.52	1.56	PASS
51	7_3_3_1_MR_HG_DX_SP	Mix	High	D_{max}	2.65	7.18	4.56	3.04	1.31	3.98	1.56	PASS
Mean of PG					2.29			2.43		3.04	1.97	PASS
					PG-18							
52	4_3_6_1_MR_HG_DX_LP	Mix	High	D_{max}	3.02	3.42	3.51	2.35	1.22	2.86	1.56	PASS
53	8_3_6_1_MR_HG_DX_LP	Mix	High	D_{max}	3.46	3.07	3.71	2.49	1.20	2.99	1.56	PASS
54	4_B_6_1_MR_HG_DX_LP	Mix	High	D_{max}	2.90	3.35	3.17	2.13	1.21	2.58	1.56	PASS
Mean of PG					3.12			2.32		2.81	1.97	PASS
					PG-21							
61	8_3_4_1_MR_LG_DX_SP	Mix	Low	D_{max}	2.82	3.28	3.37	2.25	1.19	2.68	1.56	PASS
62	4_3_4_1_MR_LG_DX_SP	Mix	Low	D_{max}	2.48	3.45	3.15	2.10	1.20	2.52	1.56	PASS
63	7_3_3_1_MR_LG_DX_SP	Mix	Low	D_{max}	3.01	6.90	4.73	3.15	1.30	4.10	1.56	PASS
Mean of PG					2.77			2.50		3.10	1.97	PASS

Table 8.2—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 3 (con.)

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
				PG-22								
64	4_3_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	2.83	2.94	3.37	2.26	1.20	2.70	1.56	PASS
65	8_3_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	2.82	3.39	3.63	2.43	1.22	2.96	1.56	PASS
66	4_B_6_1_MR_LG_DX_LP	Mix	Low	D_{max}	2.87	2.93	3.22	2.16	1.20	2.59	1.56	PASS
Mean of PG					2.84			2.28		2.75	1.96	PASS
				PG-25								
73	2_2_2_2_LR_HG_DX_SP	Low	High	D_{max}	2.07	3.33	2.62	1.75	1.19	2.08	1.56	PASS
74	3_2_1_2_LR_HG_DX_SP	Low	High	D_{max}	4.16	2.40	2.99	1.99	1.15	2.29	1.50	PASS
75	5_B_2_2_LR_HG_DX_SP	Low	High	D_{max}	3.24	2.58	2.86	1.91	1.16	2.21	1.52	PASS
Mean of PG					3.16			1.88		2.20	1.91	PASS
				PG-26								
76	4_E_6_2_LR_HG_DX_LP	Low	High	D_{max}	3.00	2.27	2.99	2.01	1.16	2.32	1.49	PASS
77	8_B_6_2_LR_HG_DX_LP	Low	High	D_{max}	3.06	2.60	3.80	2.55	1.18	2.99	1.52	PASS
78	9_3_6_2_LR_HG_DX_LP	Low	High	D_{max}	2.18	2.28	3.18	2.14	1.16	2.47	1.49	PASS
Mean of PG					2.75			2.23		2.59	1.86	PASS
				PG-29								
85	2_2_2_2_LR_LG_DX_SP	Low	Low	D_{max}	2.26	3.25	2.67	1.78	1.19	2.12	1.56	PASS
86	3_2_1_2_LR_LG_DX_SP	Low	Low	D_{max}	4.16	2.43	2.99	1.99	1.15	2.30	1.51	PASS
87	5_B_2_2_LR_LG_DX_SP	Low	Low	D_{max}	3.08	2.25	2.61	1.74	1.14	1.99	1.49	PASS
Mean of PG					3.17			1.84		2.13	1.89	PASS
				PG-30								
88	4_E_6_2_LR_LG_DX_LP	Low	Low	D_{max}	2.01	2.14	3.03	2.04	1.15	2.34	1.48	PASS
89	8_B_6_2_LR_LG_DX_LP	Low	Low	D_{max}	3.44	2.14	3.70	2.48	1.15	2.85	1.48	PASS
90	9_3_6_2_LR_LG_DX_LP	Low	Low	D_{max}	2.72	1.96	3.01	2.02	1.14	2.30	1.47	PASS
Mean of PG					2.73			2.18		2.50	1.81	PASS

Table 8.2—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 3 (con.)

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
					PG-33							
97	1_A_3_2_HR_HG_DX_SP	High	High	D_{max}	3.89	3.46	3.79	2.53	1.20	3.03	1.56	PASS
98	6_E_2_2_HR_HG_DX_SP	High	High	D_{max}	4.38	3.65	3.38	2.26	1.21	2.72	1.56	PASS
99	7_A_3_2_HR_HG_DX_SP	High	High	D_{max}	1.80	6.02	3.34	2.23	1.28	2.85	1.56	PASS
Mean of PG					3.35			2.34		2.87	1.97	PASS
					PG-34							
100	4_E_6_2_HR_HG_DX_LP	High	High	D_{max}	2.97	3.46	2.78	1.86	1.22	2.27	1.56	PASS
101	8_B_6_2_HR_HG_DX_LP	High	High	D_{max}	3.18	3.77	4.40	2.95	1.23	3.63	1.56	PASS
102	9_3_6_2_HR_HG_DX_LP	High	High	D_{max}	2.24	4.99	3.62	2.43	1.27	3.08	1.56	PASS
Mean of PG					2.80			2.41		3.00	1.97	PASS
					PG-37							
109	1_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	4.59	3.49	4.09	2.72	1.20	3.27	1.56	PASS
110	6_E_2_2_HR_LG_DX_SP	High	Low	D_{max}	3.31	3.16	2.86	1.91	1.19	2.27	1.56	PASS
111	7_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	2.06	5.41	3.18	2.12	1.26	2.67	1.56	PASS
Mean of PG					3.32			2.25		2.74	1.97	PASS
					PG-38							
112	4_E_6_2_HR_LG_DX_LP	High	Low	D_{max}	2.80	2.84	3.65	2.45	1.19	2.91	1.55	PASS
113	8_B_6_2_HR_LG_DX_LP	High	Low	D_{max}	2.95	5.01	4.47	3.00	1.27	3.81	1.56	PASS
114	9_3_6_2_HR_LG_DX_LP	High	Low	D_{max}	2.82	5.14	4.19	2.81	1.27	3.58	1.56	PASS
Mean of PG					2.85			2.75		3.43	1.96	PASS
					PG-41							
121	2_2_2_2_MR_HG_DX_SP	Mix	High	D_{max}	2.07	5.02	2.68	1.78	1.25	2.23	1.56	PASS
122	3_2_1_2_MR_HG_DX_SP	Mix	High	D_{max}	4.85	2.57	3.41	2.27	1.16	2.63	1.52	PASS
123	5_B_2_2_MR_HG_DX_SP	Mix	High	D_{max}	2.51	3.42	2.61	1.74	1.20	2.08	1.56	PASS
Mean of PG					3.14			1.93		2.32	1.93	PASS

Table 8.2—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 3 (con.)

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
					PG-42							
124	4_E_6_2_MR_HG_DX_LP	Mix	High	D_{max}	2.36	2.39	3.47	2.33	1.16	2.71	1.50	PASS
125	8_B_6_2_MR_HG_DX_LP	Mix	High	D_{max}	3.05	4.26	3.69	2.47	1.25	3.08	1.56	PASS
126	9_3_6_2_MR_HG_DX_LP	Mix	High	D_{max}	2.61	2.89	3.43	2.30	1.19	2.75	1.55	PASS
Mean of PG					2.67			2.37		2.85	1.93	PASS
					PG-45							
133	2_2_2_2_MR_LG_DX_SP	Mix	Low	D_{max}	2.32	4.66	2.74	1.83	1.24	2.27	1.56	PASS
134	3_2_1_2_MR_LG_DX_SP	Mix	Low	D_{max}	4.85	2.58	3.41	2.27	1.16	2.63	1.52	PASS
135	5_B_2_2_MR_LG_DX_SP	Mix	Low	D_{max}	2.96	3.15	2.74	1.83	1.19	2.17	1.56	PASS
Mean of PG					3.38			1.98		2.36	1.93	PASS
					PG-46							
136	4_E_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	2.89	2.40	3.59	2.41	1.16	2.81	1.50	PASS
137	8_B_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	2.96	3.71	3.67	2.46	1.23	3.03	1.56	PASS
138	9_3_6_2_MR_LG_DX_LP	Mix	Low	D_{max}	3.14	2.20	3.59	2.41	1.15	2.77	1.49	PASS
Mean of PG					3.00			2.43		2.87	1.89	PASS

[a]CMR, collapse margin ratio; SSF, spectral shape factor; ACMR, adjusted collapse margin ratio.
[b]SDC, seismic design category.

Table 8.3—Archetype seismic design properties, *R* = 4 (1 kip = 1,000 lb = 0.45 metric ton)[a]

Archetype No.	Archetype ID	Key archetype design parameters: Panel aspect ratio	Seismic design criteria: Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			Performance group (PG)-09						
25	1_E_3_1_HR_HG_DX_SP	High	High	D_{max}	0.359	0.62	13.05	53.90	1.50
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	0.265	0.56	14.48	67.20	1.50
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	0.265	0.53	6.15	24.60	1.50
			PG-10						
28	4_3_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.81	26.09	105.00	1.49
29	8_3_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.77	22.50	90.50	1.49
30	9_B_6_1_HR_HG_DX_LP	High	High	D_{max}	0.604	0.76	25.99	104.60	1.49
			PG-13						
37	1_E_3_1_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.63	11.12	46.02	1.50
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.49	13.00	61.38	1.50
39	6_D_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.51	5.36	21.40	1.50
			PG-14						
40	4_3_ 6_1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.81	21.68	87.24	1.49
41	8_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.71	18.50	74.75	1.49
42	9_B_6 _1_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.80	20.96	84.35	1.49
			PG-33						
97	1_A_3_2_HR_HG_DX_SP	High	High	D_{max}	0.359	0.64	19.49	74.40	1.50
98	6_E_2_2_HR_HG_DX_SP	High	High	D_{max}	0.265	0.57	17.39	69.60	1.50
99	7_A_3_2_HR_HG_DX_SP	High	High	D_{max}	0.359	0.62	29.71	237.70	1.50
			PG-34						
100	4_E_6_2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.88	24.65	165.30	1.49
101	8_B_6_2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.72	45.81	184.30	1.49
102	9_3_6_2_HR_HG_DX_LP	High	High	D_{max}	0.604	0.72	73.94	297.60	1.49

Table 8.3—Archetype seismic design properties, *R* = 4 (1 kip = 1,000 lb = 0.45 metric tons)[a] (con.)

		Key archetype design parameters							
			Seismic design criteria						
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
			PG-37						
109	1_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.60	16.61	63.36	1.50
110	6_E_2_2_HR_LG_DX_SP	High	Low	D_{max}	0.265	0.61	15.14	60.57	1.50
111	7_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	0.359	0.60	25.05	200.40	1.50
			PG-38						
112	4_E_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.87	20.48	137.40	1.49
113	8_B_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.73	37.66	151.57	1.49
114	9_3_6_2_HR_LG_DX_LP	High	Low	D_{max}	0.604	0.71	59.64	239.98	1.49

[a]This table presents two performance groups each including three archetypes.
[b]SDC, seismic design category.

Table 8.4—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 4

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
					Performance group (PG)-09							
25	1_E_3_1_HR_HG_DX_SP	High	High	D_{max}	3.26	4.10	3.13	2.09	1.22	2.55	1.56	PASS
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	3.08	4.23	2.48	1.65	1.22	2.02	1.56	PASS
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	4.07	3.98	2.87	1.91	1.22	2.33	1.56	PASS
Mean of PG					3.47			1.88		2.30	1.97	PASS
					PG-10							
28	4_3_6_1_HR_HG_DX_LP	High	High	D_{max}	3.43	3.75	3.63	2.43	1.21	2.95	1.56	PASS
29	8_3_6_1_HR_HG_DX_LP	High	High	D_{max}	2.62	4.32	3.45	2.31	1.22	2.82	1.56	PASS
30	9_B_6_1_HR_HG_DX_LP	High	High	D_{max}	3.16	4.20	3.53	2.37	1.22	2.89	1.56	PASS
Mean of PG					3.07			2.37		2.89	1.97	PASS
					PG-13							
37	1_E_3_1_HR_LG_DX_SP	High	Low	D_{max}	3.08	3.54	3.17	2.11	1.20	2.54	1.56	PASS
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	3.46	5.47	2.99	1.99	1.25	2.49	1.56	PASS
39	6_D_2_1_HR_LG_DX_SP	High	Low	D_{max}	4.70	3.95	3.15	2.10	1.22	2.56	1.56	PASS
Mean of PG					3.74			2.07		2.53	1.97	PASS
					PG-14							
40	4_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	3.28	3.61	3.45	2.31	1.20	2.78	1.56	PASS
41	8_3_6_1_HR_LG_DX_LP	High	Low	D_{max}	3.19	4.22	3.73	2.50	1.22	3.05	1.56	PASS
42	9_B_6_1_HR_LG_DX_LP	High	Low	D_{max}	2.60	3.85	2.99	2.01	1.21	2.43	1.56	PASS
Mean of PG					3.02			2.27		2.75	1.97	PASS
					PG-33							
97	1_A_3_2_HR_HG_DX_SP	High	High	D_{max}	3.11	3.44	2.87	1.91	1.20	2.30	1.56	PASS
98	6_E_2_2_HR_HG_DX_SP	High	High	D_{max}	2.88	4.10	2.39	1.59	1.22	1.94	1.56	PASS
99	7_A_3_2_HR_HG_DX_SP	High	High	D_{max}	4.42	5.71	3.03	2.02	1.25	2.53	1.56	PASS
Mean of PG					3.47			1.84		2.25	1.97	PASS
					PG-34							
100	4_E_6_2_HR_HG_DX_LP	High	High	D_{max}	5.25	3.19	3.57	2.39	1.18	2.83	1.56	PASS
101	8_B_6_2_HR_HG_DX_LP	High	High	D_{max}	3.24	3.91	4.06	2.72	1.21	3.29	1.56	PASS
102	9_3_6_2_HR_HG_DX_LP	High	High	D_{max}	2.34	4.42	3.70	2.48	1.23	3.05	1.56	PASS
Mean of PG					3.61			2.53		3.06	1.97	PASS

Table 8.4—Summary of static and dynamic analysis, collapse margins, and acceptance criteria, *R* = 4 (con.)

		Design configuration			Collapse margin parameters[a]						Acceptance check	
Archetype No.	Archetype ID	Panel aspect ratio	Gravity	SDC[b]	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/ fail
				PG-37								
109	1_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	3.65	3.34	3.19	2.13	1.18	2.51	1.56	PASS
110	6_E_2_2_HR_LG_DX_SP	High	Low	D_{max}	3.26	3.88	2.60	1.73	1.19	2.06	1.56	PASS
111	7_A_3_2_HR_LG_DX_SP	High	Low	D_{max}	5.19	5.27	3.27	2.18	1.25	2.73	1.56	PASS
Mean of PG					4.03			2.01		2.43	1.97	PASS
				PG-38								
112	4_E_6_2_HR_LG_DX_LP	High	Low	D_{max}	4.19	2.91	3.37	2.26	1.18	2.66	1.56	PASS
113	8_B_6_2_HR_LG_DX_LP	High	Low	D_{max}	3.32	3.77	4.14	2.78	1.20	3.33	1.56	PASS
114	9_3_6_2_HR_LG_DX_LP	High	Low	D_{max}	2.71	4.97	3.80	2.55	1.25	3.19	1.56	PASS
Mean of PG					3.41			2.53		3.06	1.97	PASS

[a]CMR, collapse margin ratio; SSF, spectral shape factor; ACMR, adjusted collapse margin ratio.
[b]SDC, seismic design category.

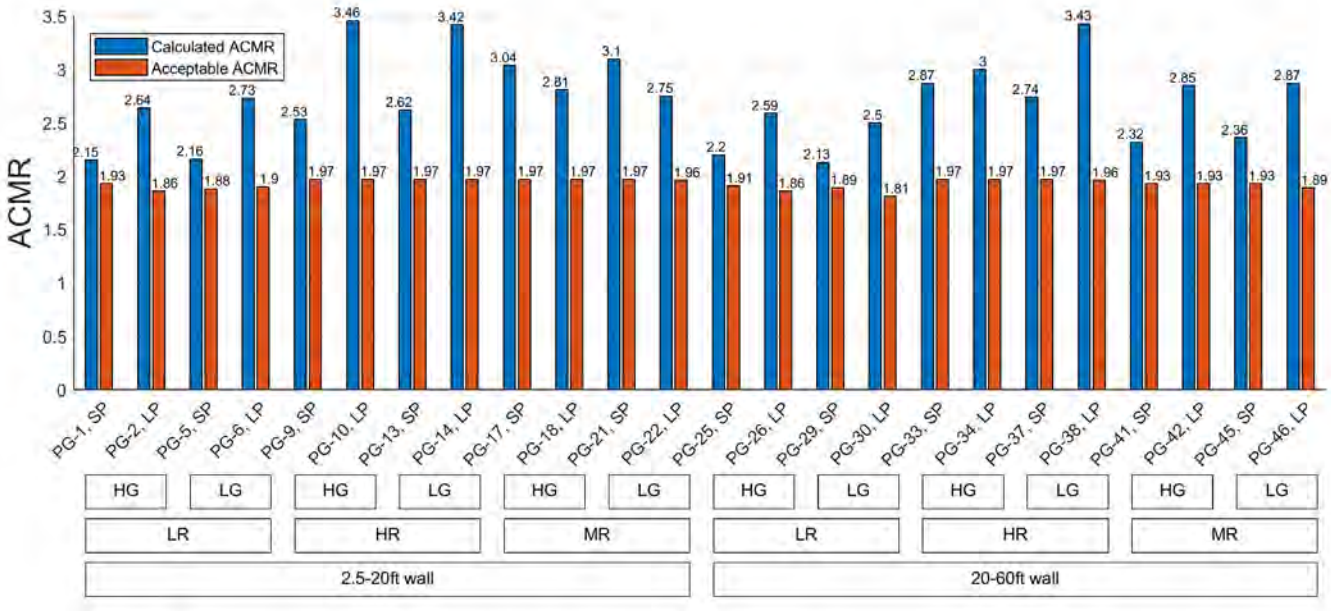


Figure 8.1. Summary of adjusted collapse margin ratio (ACMR) for performance groups (PG), R = 3, ξ = 0.02, ϕ = 0.50, nonsimulated collapse = 4.5% interstory drift for low aspect ratio (LR) and mixed aspect ratio (MR) and 5.5% for high aspect ratio (HR) (SP, short period; LP, long period; HG high gravity; LG, low gravity) (1 ft = 0.3 m).

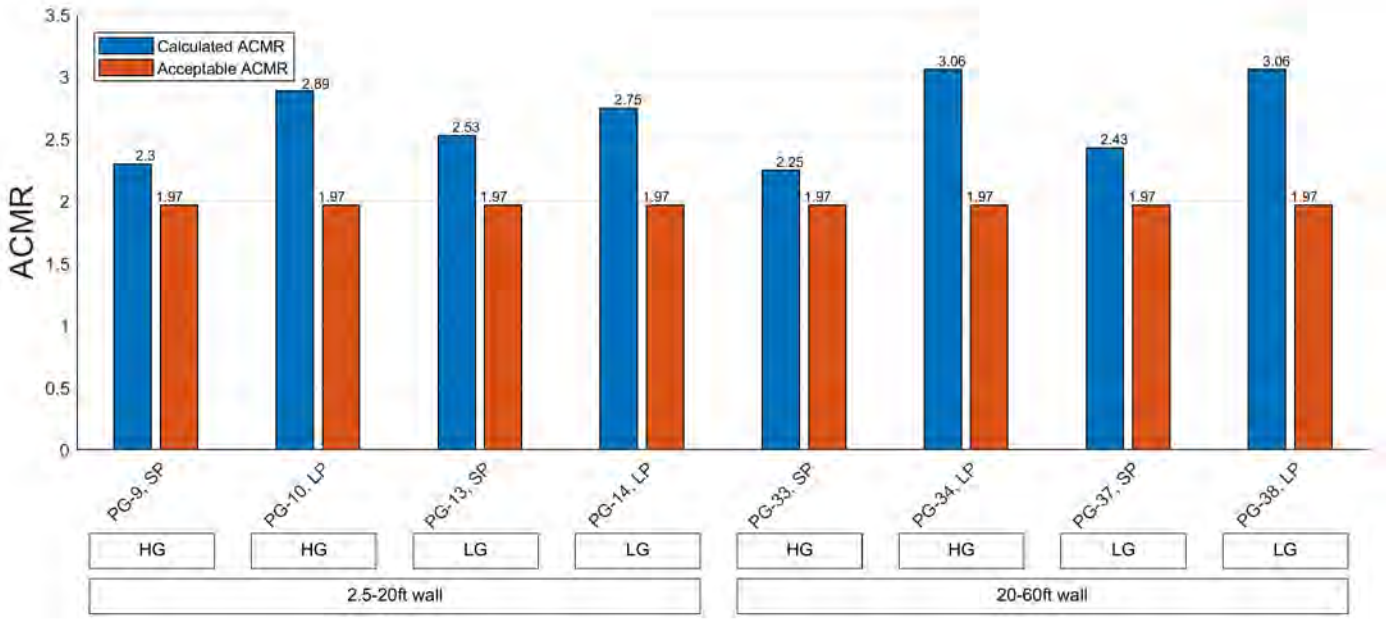


Figure 8.2. Summary of adjusted collapse margin ratio (ACMR) for high aspect ratio (HR) panel performance groups (PG), R = 4, ξ = 0.02, ϕ = 0.50, nonsimulated collapse = 5.5% interstory drift for HR (SP, short period; LP, long period; HG, high gravity; LG, low gravity) (1 ft = 0.3 m).

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Appendix A—Gravity Load and Seismic Weight

Figure A1 and Table A1 present typical composition of the exterior wall, interior wall, floor, and roof materials and the corresponding unit weights that are used to calculate the seismic weight of the structure. These assigned layers are for the purpose of this study only and are not verified for fire and sound performance.

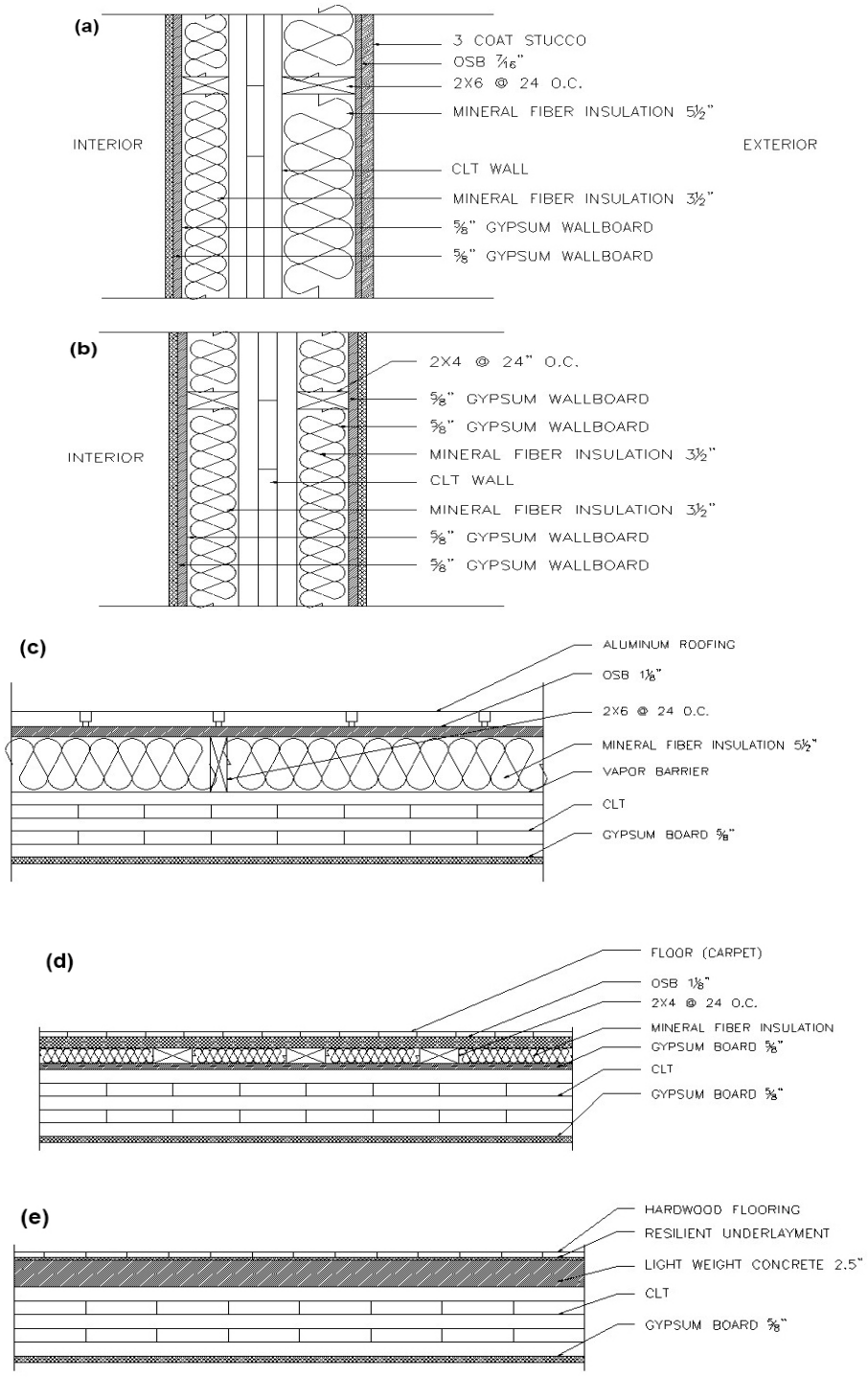


Figure A1. Wall and floor typical composition for weight calculations (a) exterior wall; (b) interior wall; (c) roof; (d) floor (light); (e) floor (heavy).

Table A1—Dead load for structural and nonstructural components

Material	Weight (lb/ft^2)	Reference
Exterior wall		
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Gypsum wallboard 5/8 in.	2.60	"
2 by 4 @ 24 in. o.c.	0.61	NDS supplement equation for density
Mineral fiber insulation 3.5 in.	2.3	Auralex technical manual
Mineral fiber insulation 5.5 in.	3.67	Auralex technical manual
2 by 6 @ 24 in. o.c.	0.96	NDS supplement equation for density
OSB 7/16 in.	1.3125	Table C3-1
Stucco 7/8 in.	10	
Wall 4.125 in. (three layers of 1.375 in.)	9.625	NDS supplement equation for density
Interior wall		
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Gypsum wallboard 5/8 in.	2.60	"
2 by 4 @ 24 in. o.c.	0.61	NDS supplement equation for density
Mineral fiber insulation 3.5 in.	2.3	Auralex technical manual
Mineral fiber insulation 3.5 in.	2.3	Auralex technical manual
2 by 4 @ 24 in. o.c.	0.61	NDS supplement equation for density
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Gypsum wallboard 5/8 in.	2.60	"
Wall 4.125 in. (three layers of 1.375 in.)	9.625	NDS supplement equation for density
Floor (light)		
Floor (carpet)	2	
OSB 1 1/8 in.	3.375	Table C3-1 ASCE 7
2 by 4 @ 24 in. o.c.	0.61	NDS supplement equation for density
Mineral fiber insulation 1.5 in.	1.0	Auralex technical manual
Gypsum board 5/8 in.	2.60	Table C3-2 ASCE 7
Floor CLT 9.275 in.	21.64	NDS supplement equation for density
Floor (heavy)		
Hardwood flooring	4	Table C3-1 ASCE 7
Light weight concrete 2.5 in.	20	Table C3-1 ASCE 7
Resilient underlayment	1.1	Kinetics noise control specs and Table C3-1 ASCE 7
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Floor CLT 9.275 in.	21.64	NDS supplement equation for density
Roof		
Deck, metal 20 gage	2.5	Table C3-1 ASCE 7
OSB 1 1/8 in.	3.375	Table C3-1 ASCE 7
2 by 6 @ 24 in. o.c.	0.96	NDS supplement equation for density
Mineral fiber insulation 5.5 in.	3.67	Auralex technical manual
Vapor barrier	0.7	Table C3-1 ASCE 7
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Floor CLT 6.625 in.	15.46	NDS supplement equation for density

Appendix B—Index Buildings

In-plane deflection for shear wall archetypes in this appendix do not include effects of panel shear deformations, which are typically less than 5% of the total in-plane deflection of the shear wall. Exclusion of this component of shear wall deflection is considered to have no practical effect on changing archetype designs and adjusted collapse margin ratio (ACMR) results.

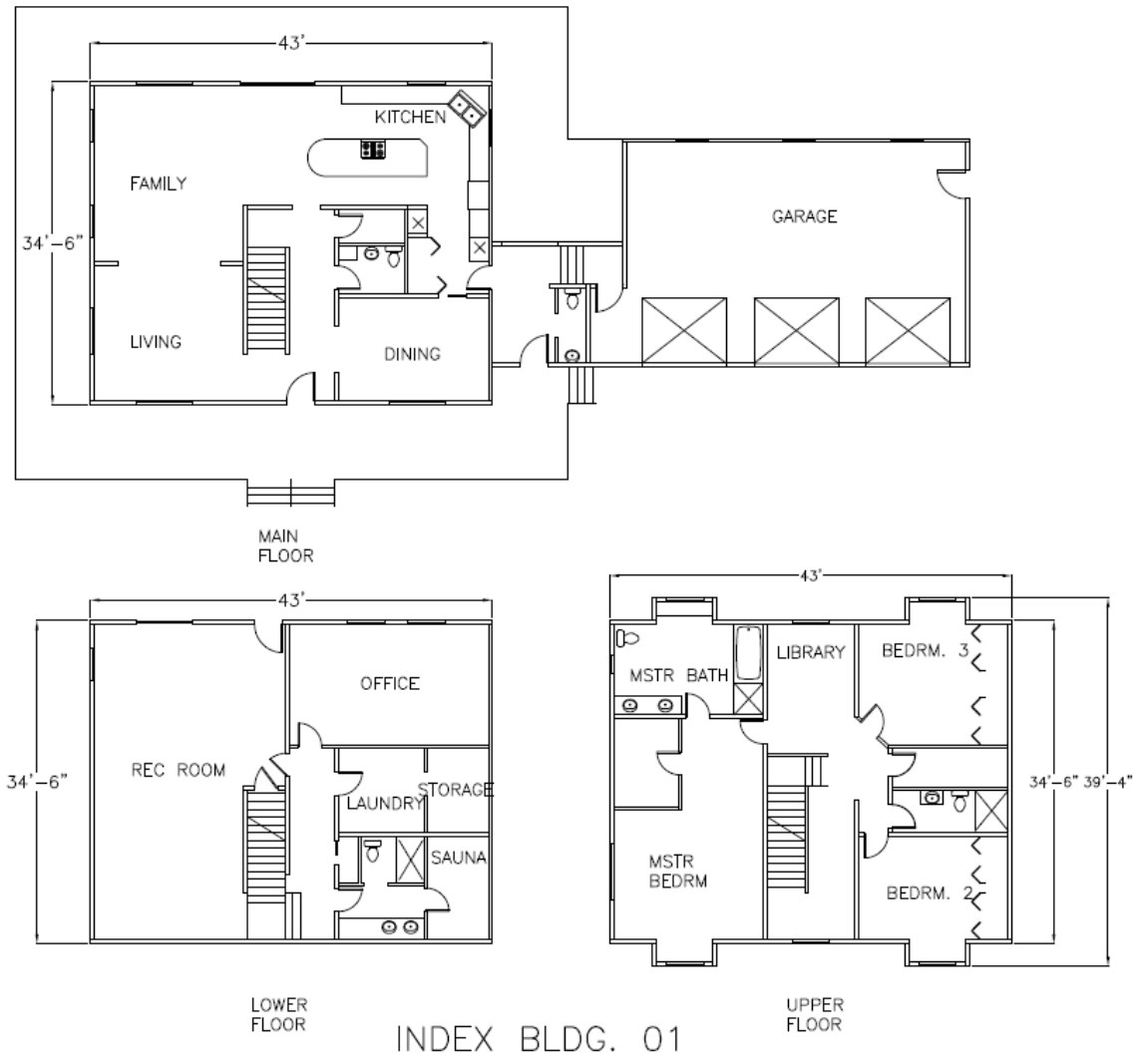


Figure B1. Index Building 1 floor plan.

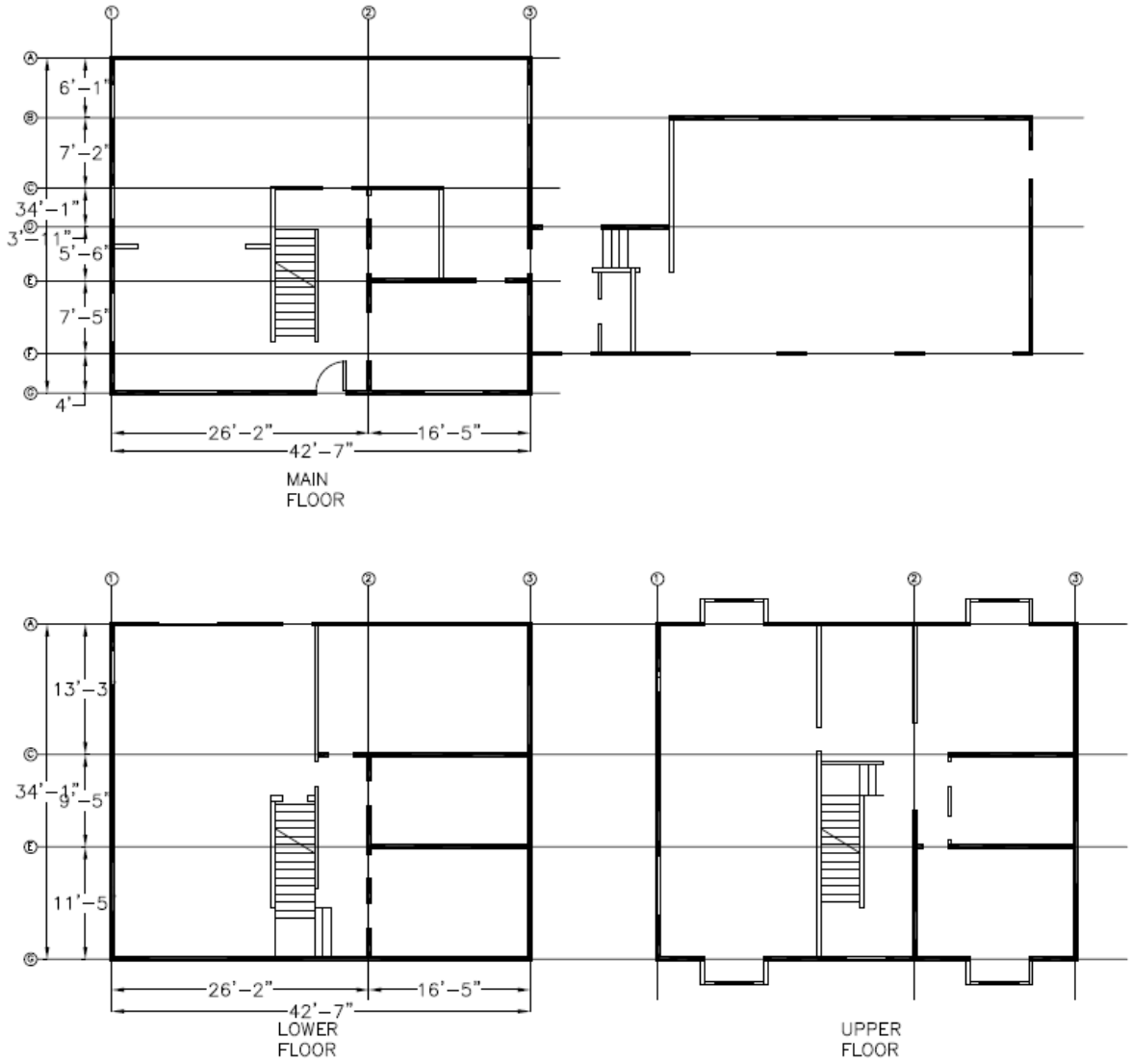


Figure B2. Index Building 1 assigned shear walls.

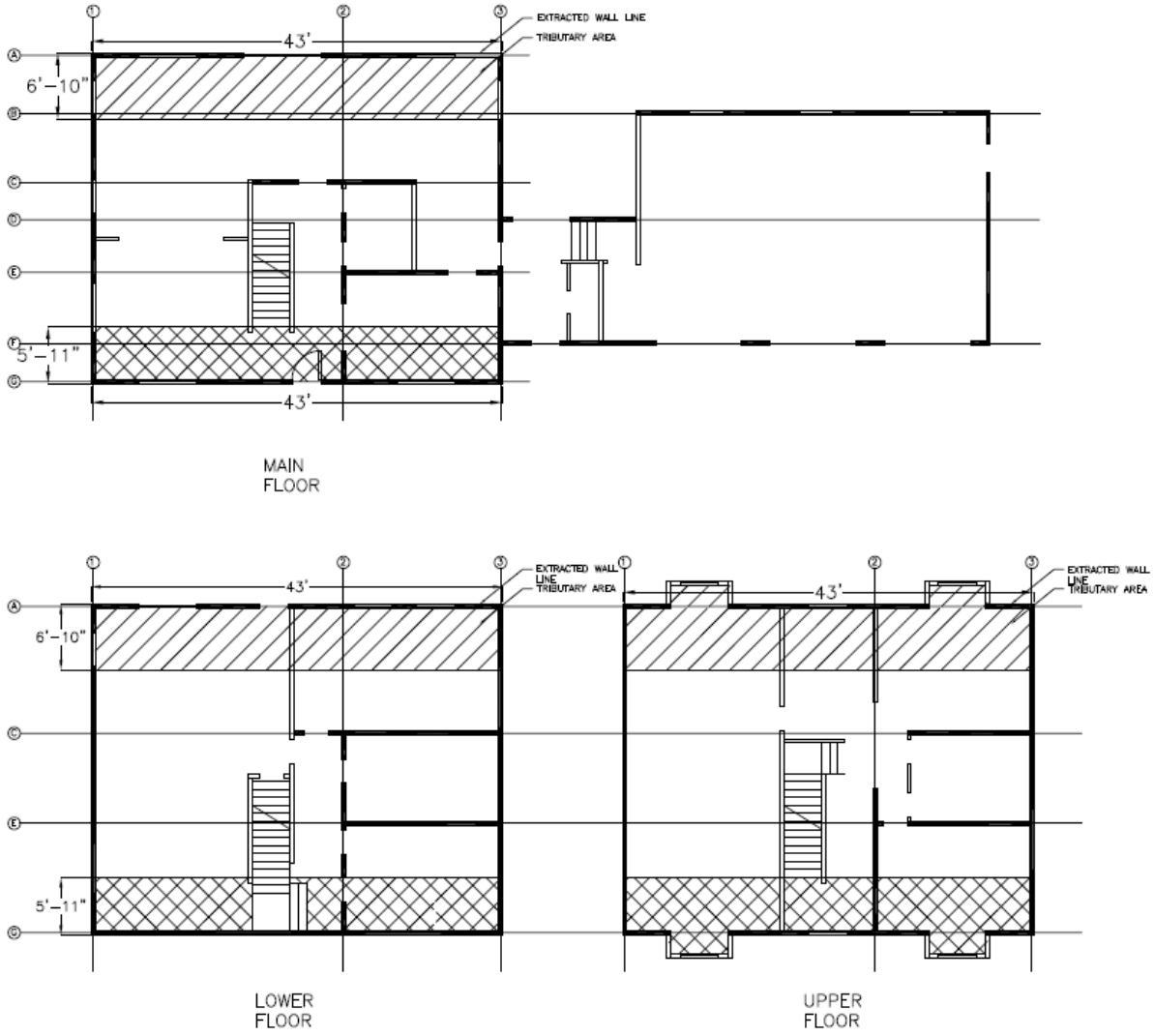


Figure B3. Index Building 1 tributary area.

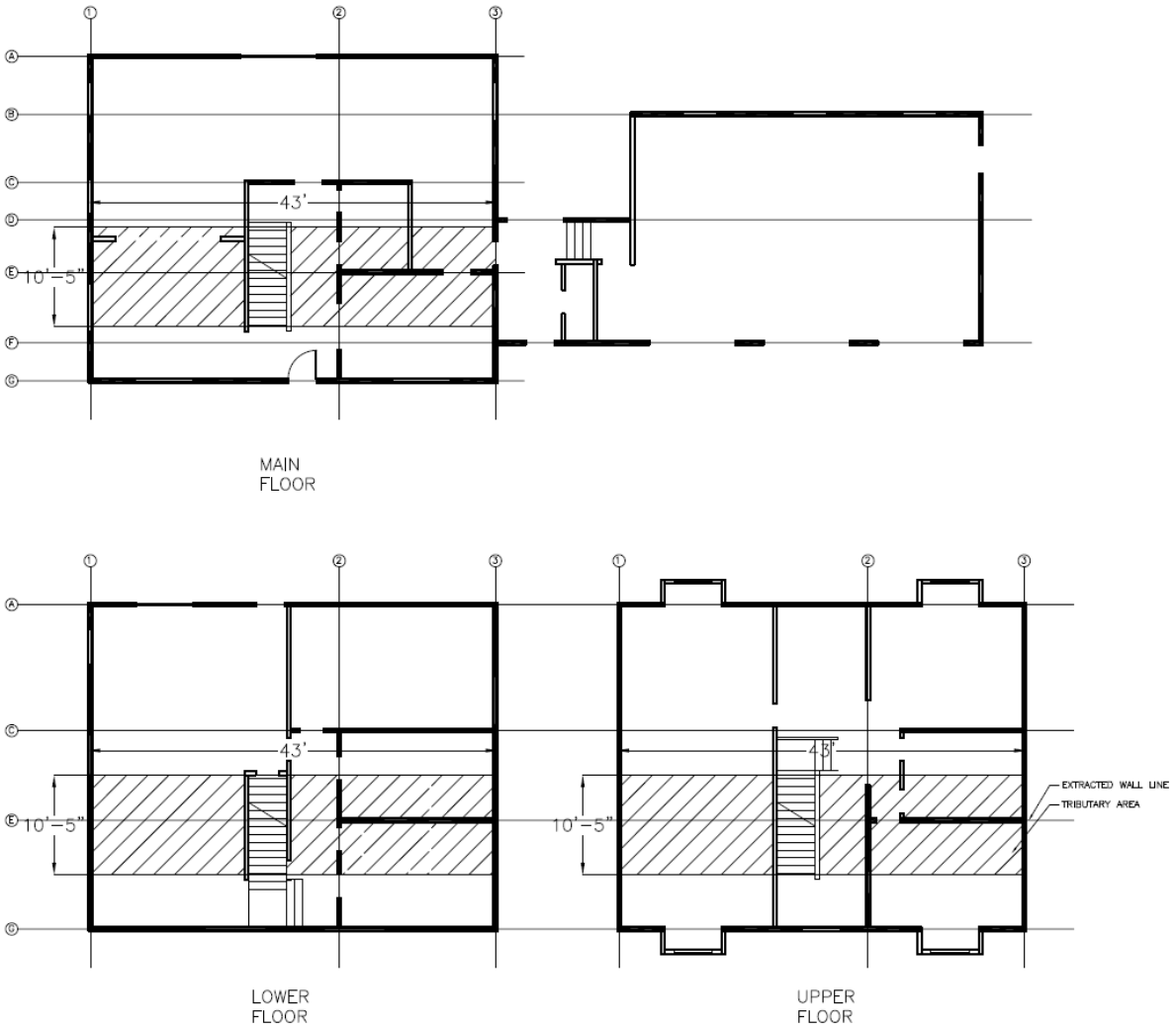


Figure B4. Index Building 1 tributary area.

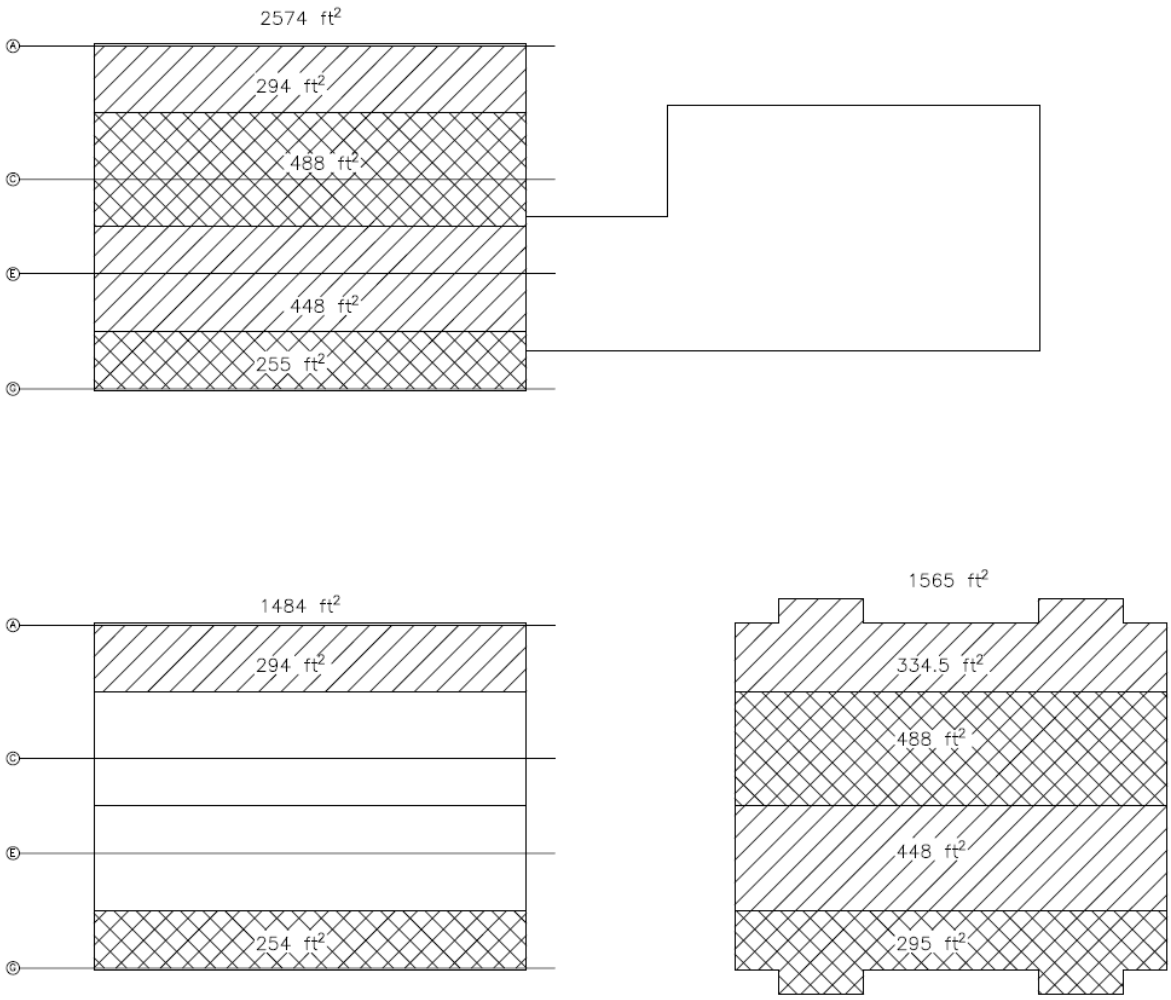
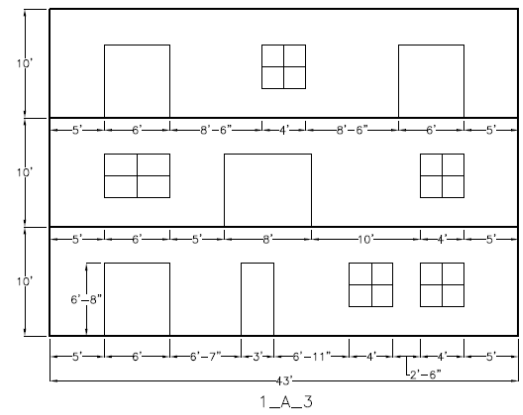
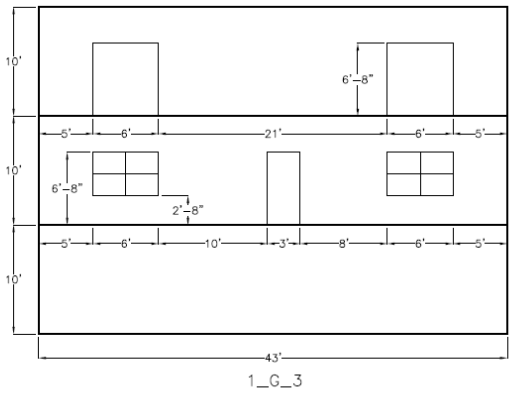


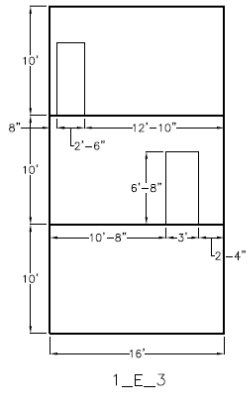
Figure B5. Index Building 1 tributary area.



1_A_3



1_G_3



1_E_3

Figure B6. Index Building 1 extracted shear wall lines.

Table B1—Extracted archetypes from Index Building 1

Group No.	Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-1	2.5ft-20ft wall, Low aspect ratio panels	High	SDC D_{max}	1_E_3_1_LR_HG_DX_SP	1
PG-5		Low		1_E_3_1_LR_LG_DX_SP	13
PG-9	2.5ft-20ft wall, High aspect ratio panels	High		1_E_3_1_HR_HG_DX_SP	25
PG-13		Low		1_E_3_1_HR_LG_DX_SP	37
PG-33	20ft-60ft wall, High aspect ratio panels	High		1_A_3_2_ HR_HG_ DX_SP	97
PG-37		Low		1_A_3_2_ HR_LG_ DX_SP	109

Table B2—Seismic weight detailed calculation, Index Building 1, 3-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			1585					46387.7			**81981**	46387.7			**81981**
	3	10		163.01	1474.07	87.83	828		49707	21481			49707	21481	
2			1484					46350.3			**114780**	73229.2			**141659**
	2	10		153.34	1304.73	94.25	836		43997	21676			43997	21676	
1			1483					46319.0			**114206**	73179.9			**141067**
	1	10		153.34	1409.15	95.42	871		47518	22584			47518	22584	
Ground			0												

Table B3—Seismic weight summary for Index Building 1

Level	*h* (ft)	W_{Level} (kip), light	W_{Level} (kip), heavy
Roof		82.0	82.0
	10		
2		114.8	141.7
	10		
1		114.2	141.1
	10		
Ground			

INPUT DATA

Total Height	h_n=	30.0	ft				
Total Weight	W=	311	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.3589					
	Ts=	0.6					
	Cs=	0.333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	5,898					

DESIGN SUMMARY

Total base shear

V = 103.67

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	82	2,460	0.417	43.2		
	10.00						43.2	
2		20.0	115	2,296	0.389	40.4		432
	10.00						83.6	
1		10.0	114	1,142	0.194	20.1		1,268
	10.00						103.7	
		0.0						2,305

Seismic base shear calculation, low gravity, Index Building 1, 3-story, R = 3

INPUT DATA

Total Height	h_n=	30.0	ft				
Total Weight	W=	365	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.3589					
	Ts=	0.6					
	Cs=	0.333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	6,705					

DESIGN SUMMARY

Total base shear

V = 121.60

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	82	2,460	0.367	44.6		
	10.00						44.6	
2		20.0	142	2,834	0.423	51.4		446
	10.00						96.0	
1		10.0	141	1,411	0.210	25.6		1,406
	10.00						121.6	
		0.0						2,622

Seismic base shear calculation, high gravity, Index Building 1, 3-story, R = 3

Table B4—Tributary load calculation, Index Building 1, 3-story, E-W direction, R = 3*

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 3	**A**	**294.00**	**0.198**	43.24	**8.56**	44.65	**8.84**
	C	488.00	0.329				
	E	**448.00**	**0.302**		**13.04**		**13.47**
	G	255.00	0.172				
Story 2	**A**	**294.00**	**0.198**	83.59	**16.55**	96.08	**19.02**
	C	488.00	0.329				
	E	**448.00**	**0.302**		**25.22**		**28.99**
	G	255.00	0.172				
Story 1	**A**	**334.50**	**0.214**	103.67	**22.15**	121.80	**26.02**
	C	488.00	0.312				
	E	**448.00**	**0.286**		**29.67**		**34.86**
	G	295.00	0.188				

* Seismic load for shear wall line E is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B5—Archetype 1 (1_E_3_1_LR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	4.13	3	A3	3	D	1563.0	4	5/8	0.63	1.19	1346.9	7.82	6.73	1.16
2	1	S	5	4.125	3	A3	6	D	3126.0	4	1	0.88	1.84	2898.6	15.63	14.49	1.08
1	1	S	5	6.875	5	A3	7	D	3647.0	4	1 3/8	0.88	2.21	3485.6	18.24	17.43	1.05

*For unreinforced option and values shown are before applying C_d

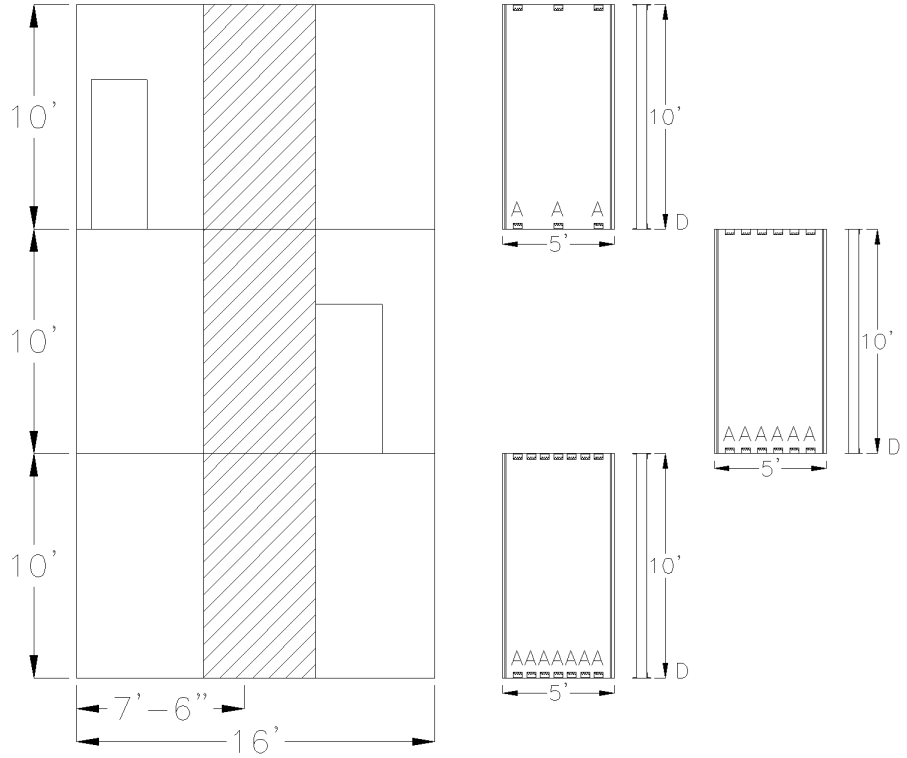


Figure B7. Archetype 01.

Table B6—Archetype 13 (1_E_3_1_LR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	4.125	3	A3	3	D	1563.0	4	5/8	0.62	1.17	1304.4	7.82	6.52	1.20
2	1	S	5	4.125	5	A3	5	D	2605.0	4	1	0.82	1.71	2521.9	13.03	12.61	1.03
1	1	S	5	6.875	5	A3	6	D	3126.0	4	1 1/4	0.82	2.01	2966.6	15.63	14.83	1.05

*Values shown are for unreinforced option and before applying C_d

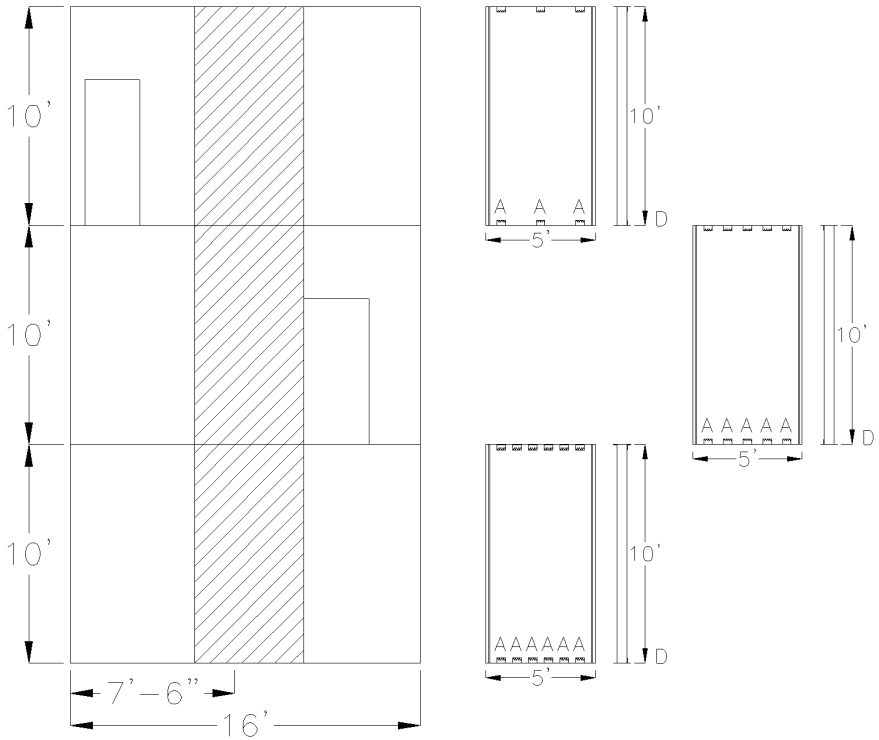


Figure B8. Archetype 13.

Table B7—Archetype 25 (1_E_3_1_HR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.73	1.02	898.0	7.82	6.73	1.16
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/4	0.92	1.75	1932.4	15.63	14.49	1.08
1	3	M	2.5	6.875	5	A3	3	D	3126.0	2	1 3/4	0.81	2.40	2323.7	23.45	17.43	1.35

*Values shown are for unreinforced option and before applying C_d

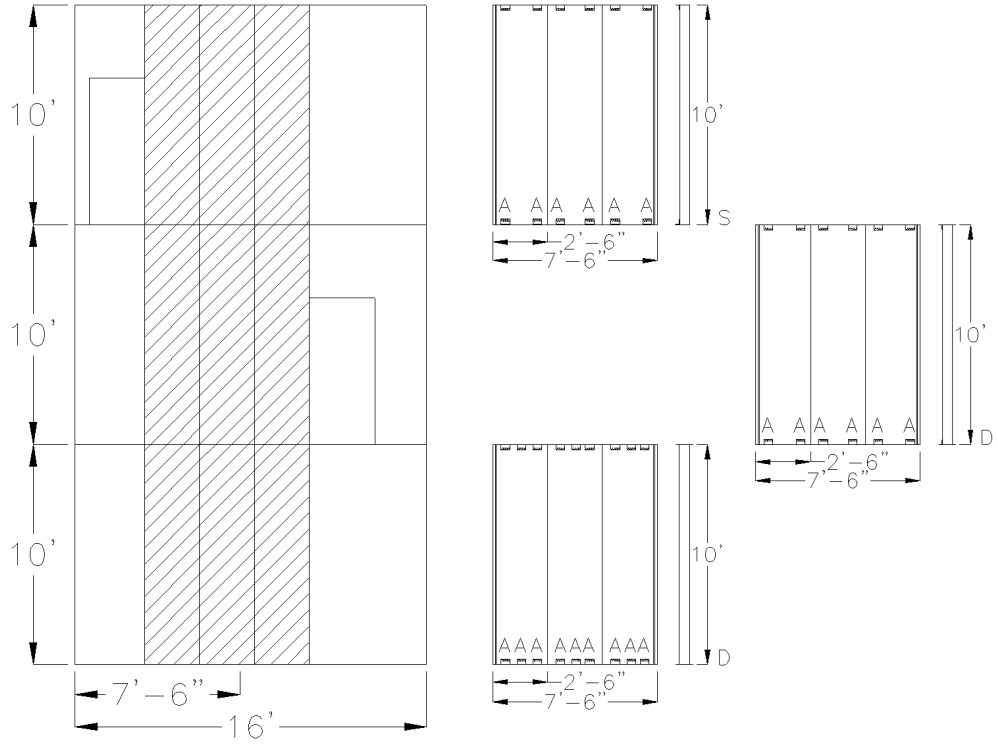


Figure B9. Archetype 25.

Table B8—Archetype 37 (1_E_3_1_HR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.72	1.01	869.6	7.82	6.52	1.20
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/4	0.83	1.68	1681.3	15.63	12.61	1.24
1	3	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.83	1.98	1977.8	15.63	14.83	1.05

*Values shown are for unreinforced option and before applying C_d

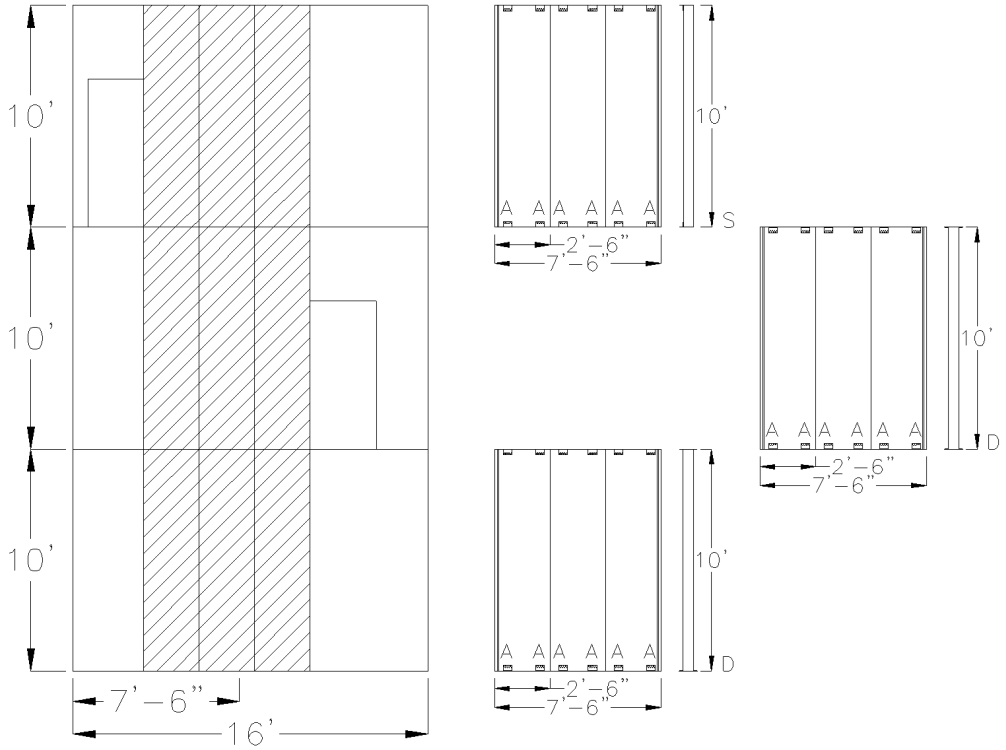


Figure B10. Archetype 37.

Table B9—Archetype 97 (1_A_3_2_ HR_HG_ DX_SP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.73	0.45	589.3	5.21	2.95	1.77
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/8	0.88	0.80	1268.4	10.42	6.34	1.64
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.93	1.03	1734.7	10.42	8.67	1.20

*There are three of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

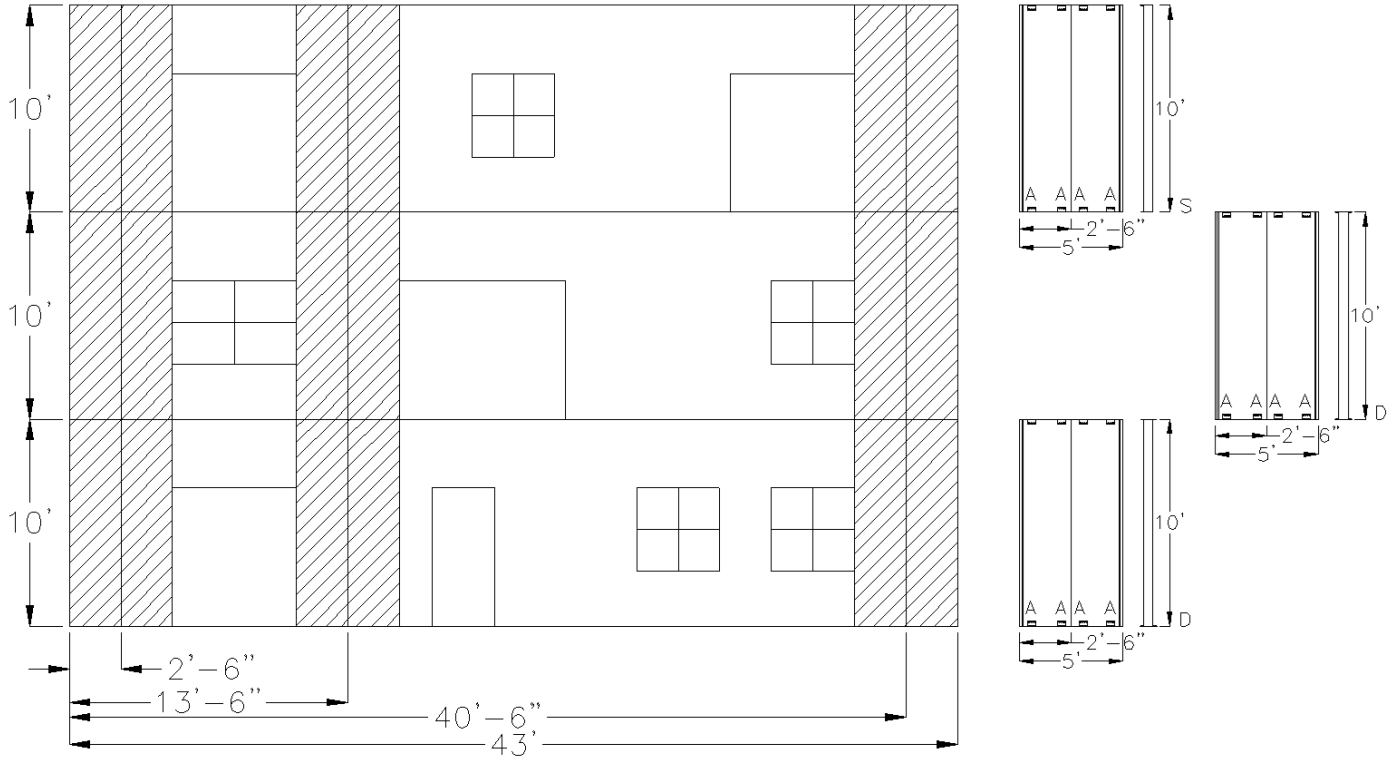


Figure B11. Archetype 97.

Table B10—Archetype 109 (1_A_3_2_ HR_LG_ DX_SP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.72	0.44	570.7	5.21	2.85	1.83
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/8	0.81	0.75	1103.3	10.42	5.52	1.89
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.85	0.96	1476.8	10.42	7.38	1.41

*There are three of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

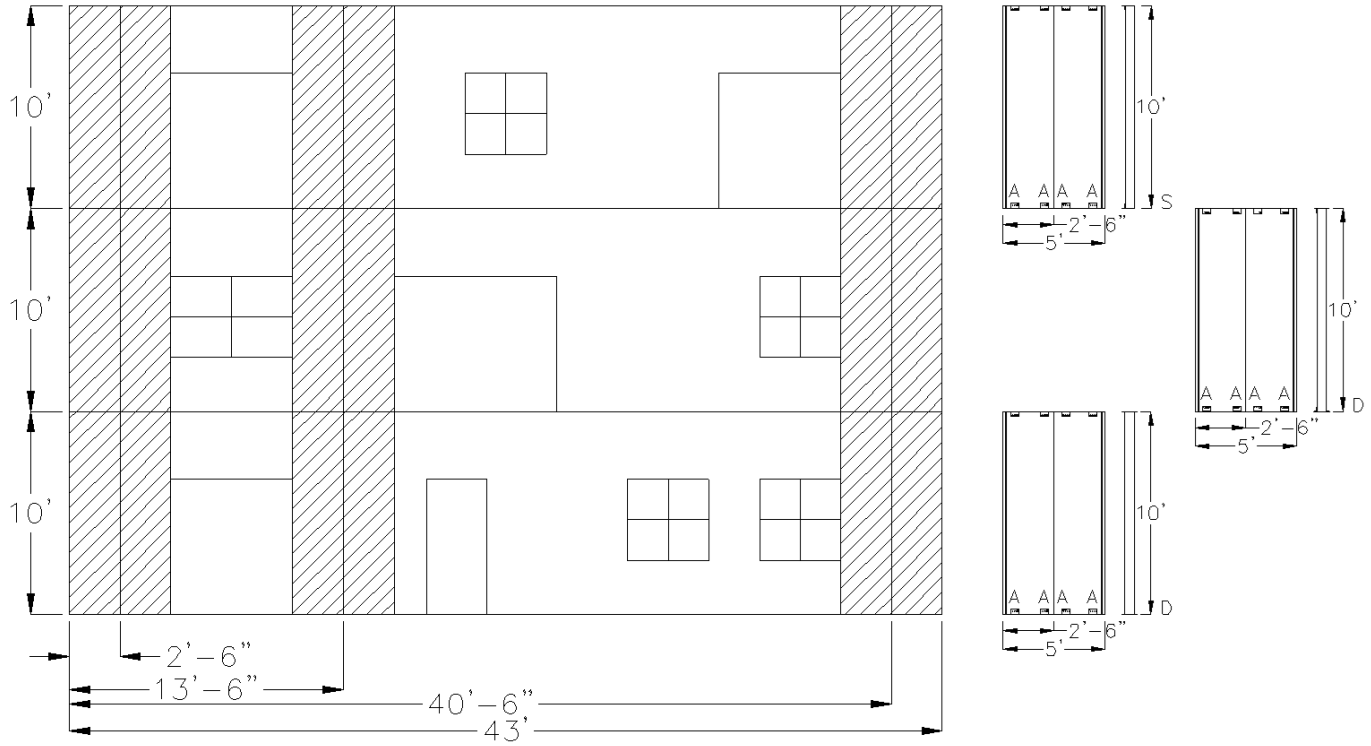


Figure B12. Archetype 109.

INPUT DATA

Total Height	h_n=	30.0	ft					
Total Weight	W=	311	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	4						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.3589						
	Ts=	0.6						
	Cs=	0.250						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	5,898						

DESIGN SUMMARY

Total base shear V = 77.75

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	82	2,460	0.417	32.4		
	10.00						32.4	
2		20.0	115	2,296	0.389	30.3		324
	10.00						62.7	
1		10.0	114	1,142	0.194	15.1		951
	10.00						77.8	
		0.0						1,729

Seismic base shear calculation, low gravity, Index Building 1, 3-story, R = 4

INPUT DATA

Total Height	h_n=	30.0	ft					
Total Weight	W=	365	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	4						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.3589						
	Ts=	0.6						
	Cs=	0.250						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	6,705						

DESIGN SUMMARY

Total base shear V = 91.20

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	82	2,460	0.367	33.5		
	10.00						33.5	
2		20.0	142	2,834	0.423	38.5		335
	10.00						72.0	
1		10.0	141	1,411	0.210	19.2		1,055
	10.00						91.2	
		0.0						1,967

Seismic base shear calculation, high gravity, Index Building 1, 3-story, R = 4

Table B11—Tributary load calculation, Index Building 1, 3-story, E-W direction, R = 4*

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 3	**A**	**294.00**	**0.198**	32.43	**6.42**	33.46	**6.62**
	C	488.00	0.329				
	E	**448.00**	**0.302**		**9.78**		**10.09**
	G	255.00	0.172				
Story 2	**A**	**294.00**	**0.198**	62.7	**12.41**	72.0	**14.26**
	C	488.00	0.329				
	E	**448.00**	**0.302**		**18.91**		**21.72**
	G	255.00	0.172				
Story 1	**A**	**334.50**	**0.214**	77.75	**16.61**	91.20	**19.49**
	C	488.00	0.312				
	E	**448.00**	**0.286**		**22.25**		**26.10**
	G	295.00	0.188				

* Seismic load for shear wall line E is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B12—Archetype 25 (1_E_3_1_HR_HG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.61	0.92	673.0	7.82	5.05	1.55
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	1	0.84	1.43	1448.2	11.72	10.86	1.08
1	3	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.75	1.93	1739.9	15.63	13.05	1.20

*Values shown are for unreinforced option and before applying C_d.

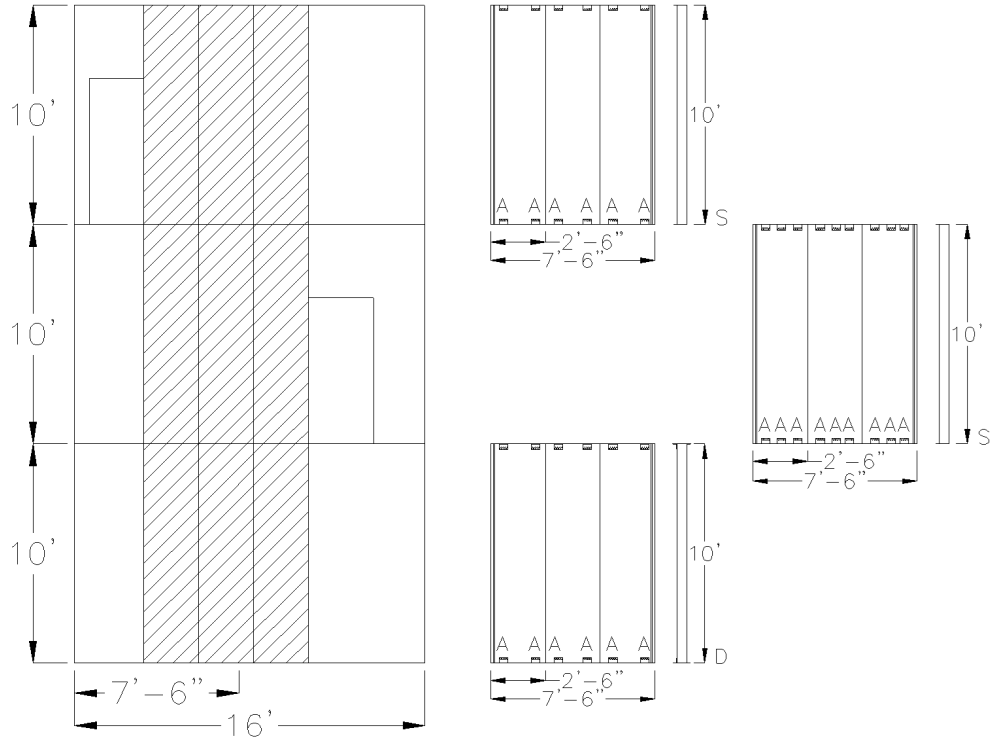


Figure B13. Archetype 25.

Table B13—Archetype 37 (1_E_3_1_HR_LG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.60	0.91	652.2	7.82	4.89	1.60
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	1	0.77	1.37	1260.9	11.72	9.46	1.24
1	3	M	2.5	6.875	5	A3	3	S	1563.0	2	1 3/8	0.76	1.63	1483.3	11.72	11.12	1.05

*Values shown are for unreinforced option and before applying C_d.

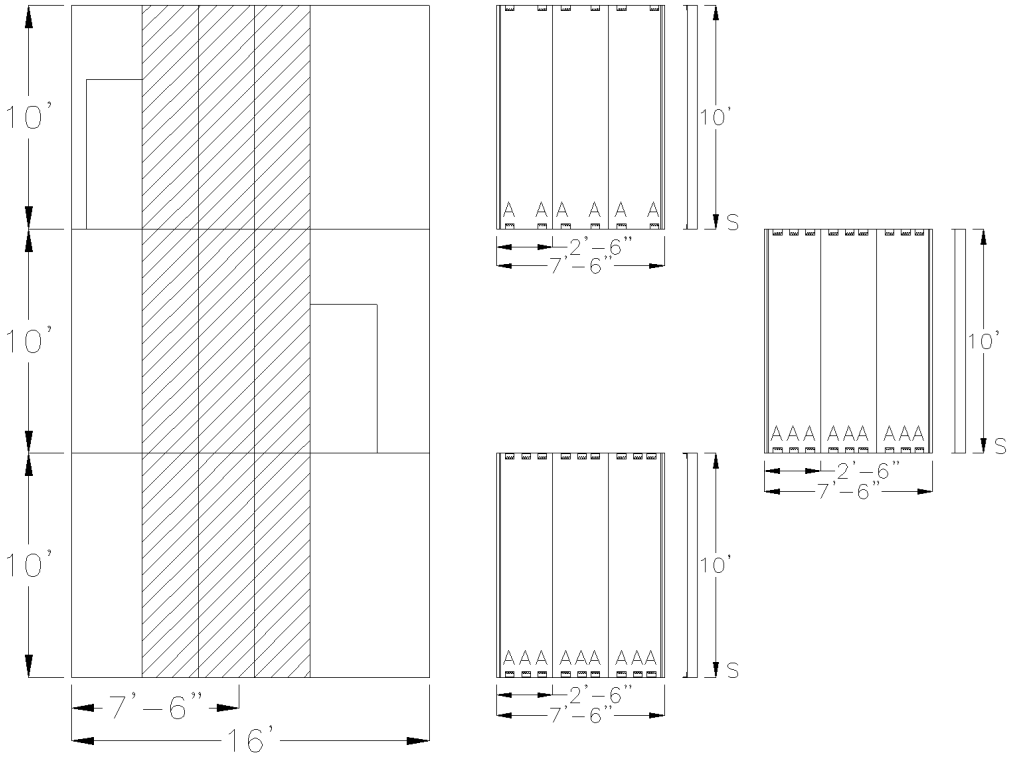


Figure B14. Archetype 37.

Table B14—Archetype 97 (1_A_3_2_ HR_HG_ DX_SP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.64	0.39	441.6	5.21	2.21	2.36
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.91	0.58	950.4	5.21	4.75	1.10
1	2	M	2.5	6.875	5	A3	3	S	1563.0	2	1 1/4	0.88	0.82	1299.1	7.82	6.50	1.20

*There are three of (2)2.5-ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

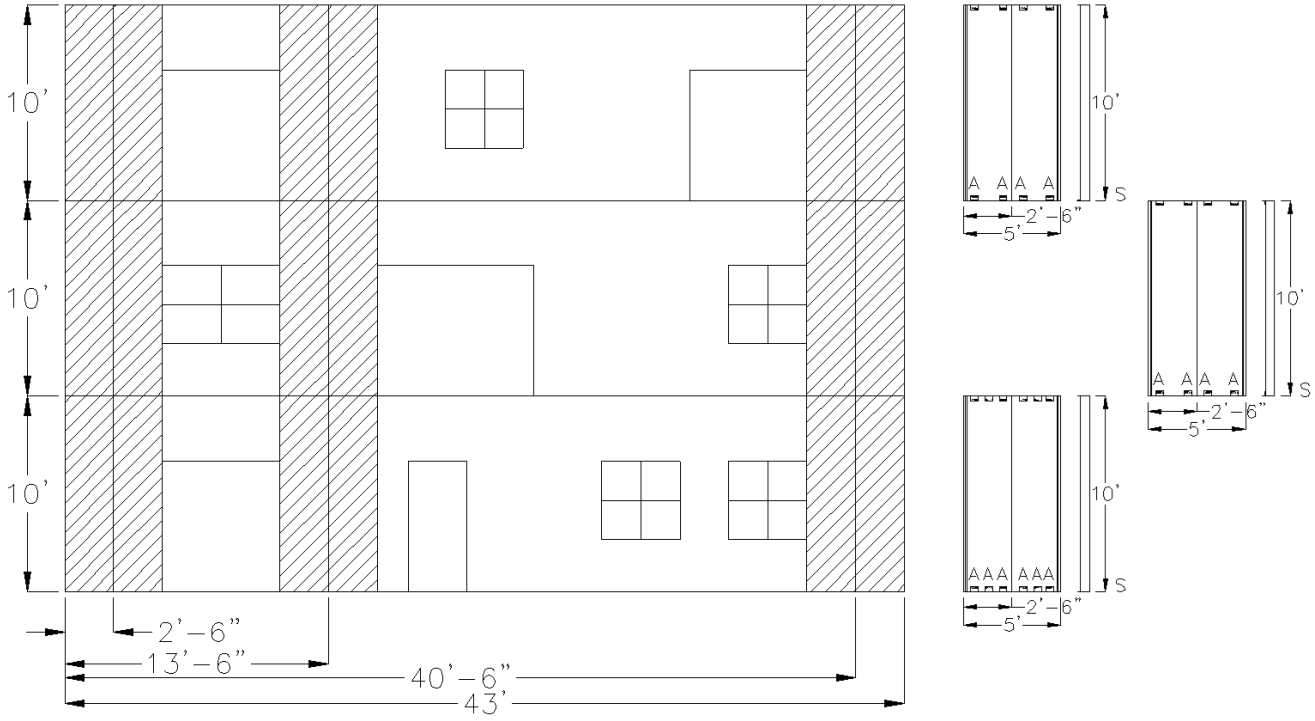


Figure B15. Archetype 97.

Table B15—Archetype 109 (1_A_3_2_ HR_LG_ DX_SP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.63	0.38	428.0	5.21	2.14	2.43
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.85	0.54	827.5	5.21	4.14	1.26
1	2	M	2.5	4.125	3	A3	3	S	1563.0	2	1 1/4	0.90	0.69	1107.8	7.82	5.54	1.41

*There are three of (2)2.5-ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

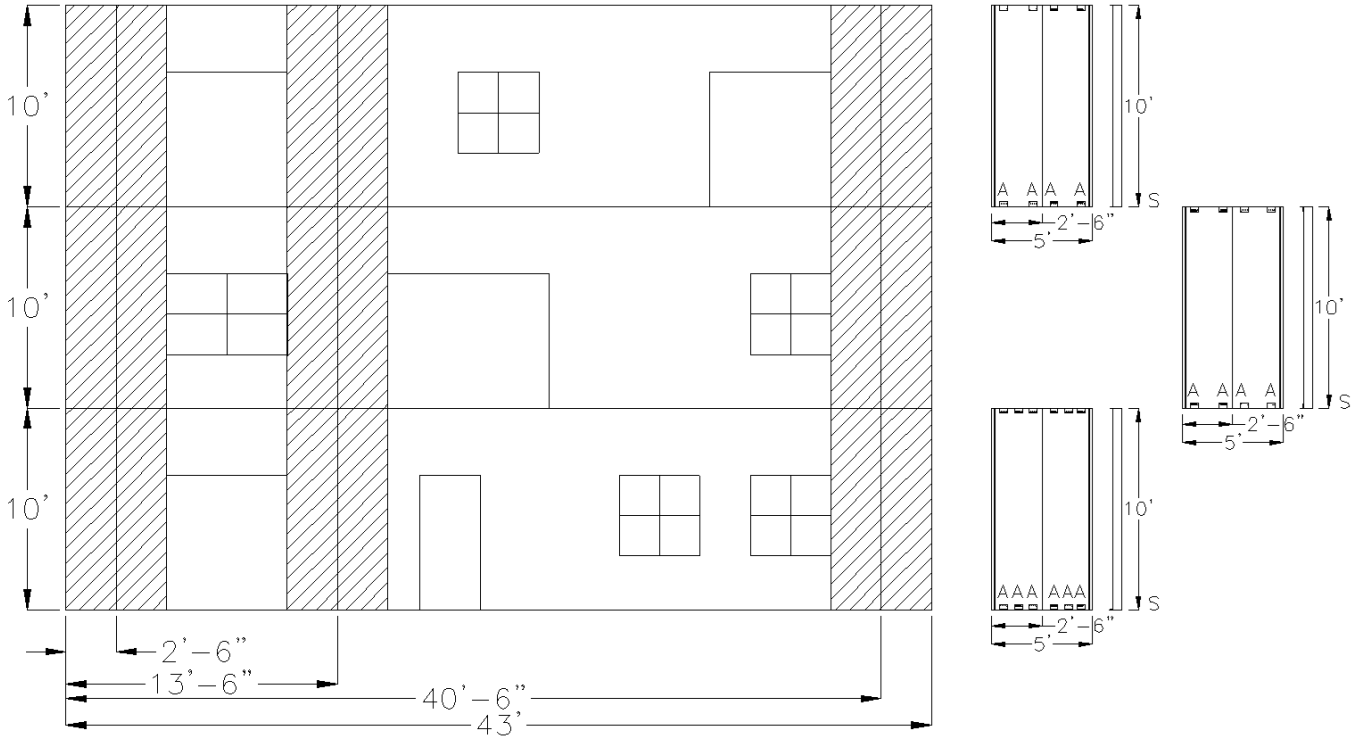


Figure B16. Archetype 109.

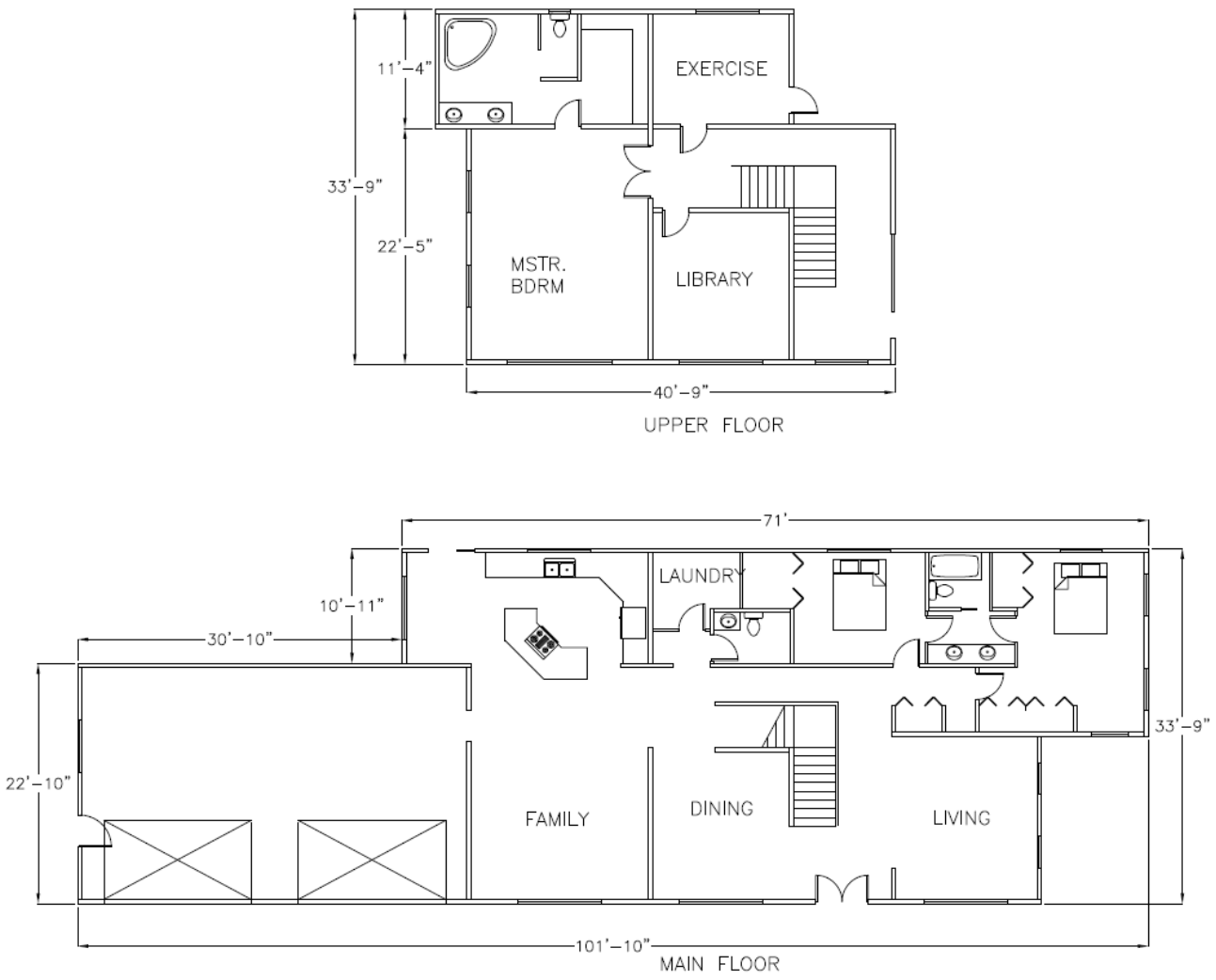


Figure B17. Index Building 2 floor plan.

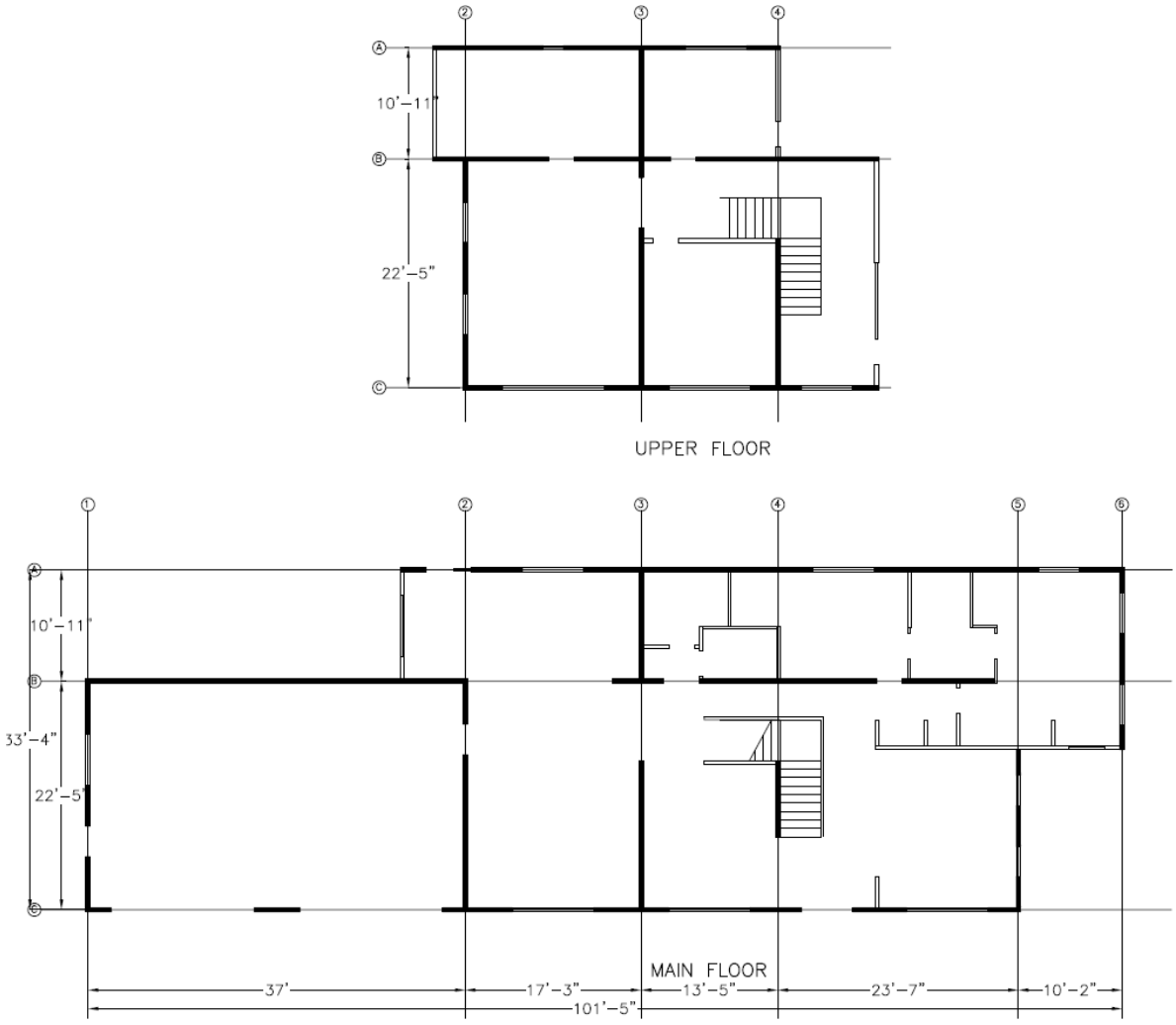


Figure B18. Index Building 2 assigned shear walls.

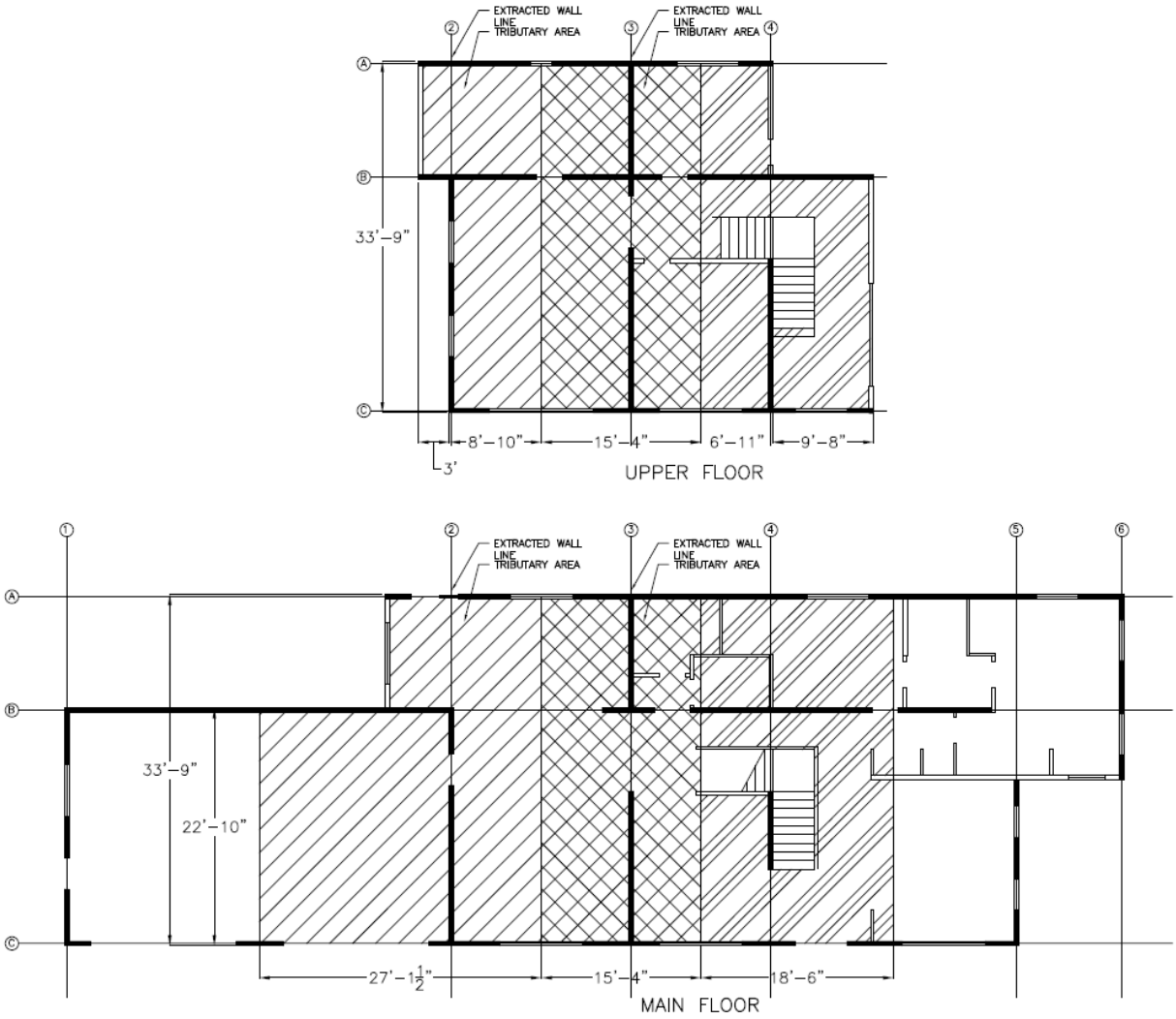


Figure B19. Index Building 2 tributary area.

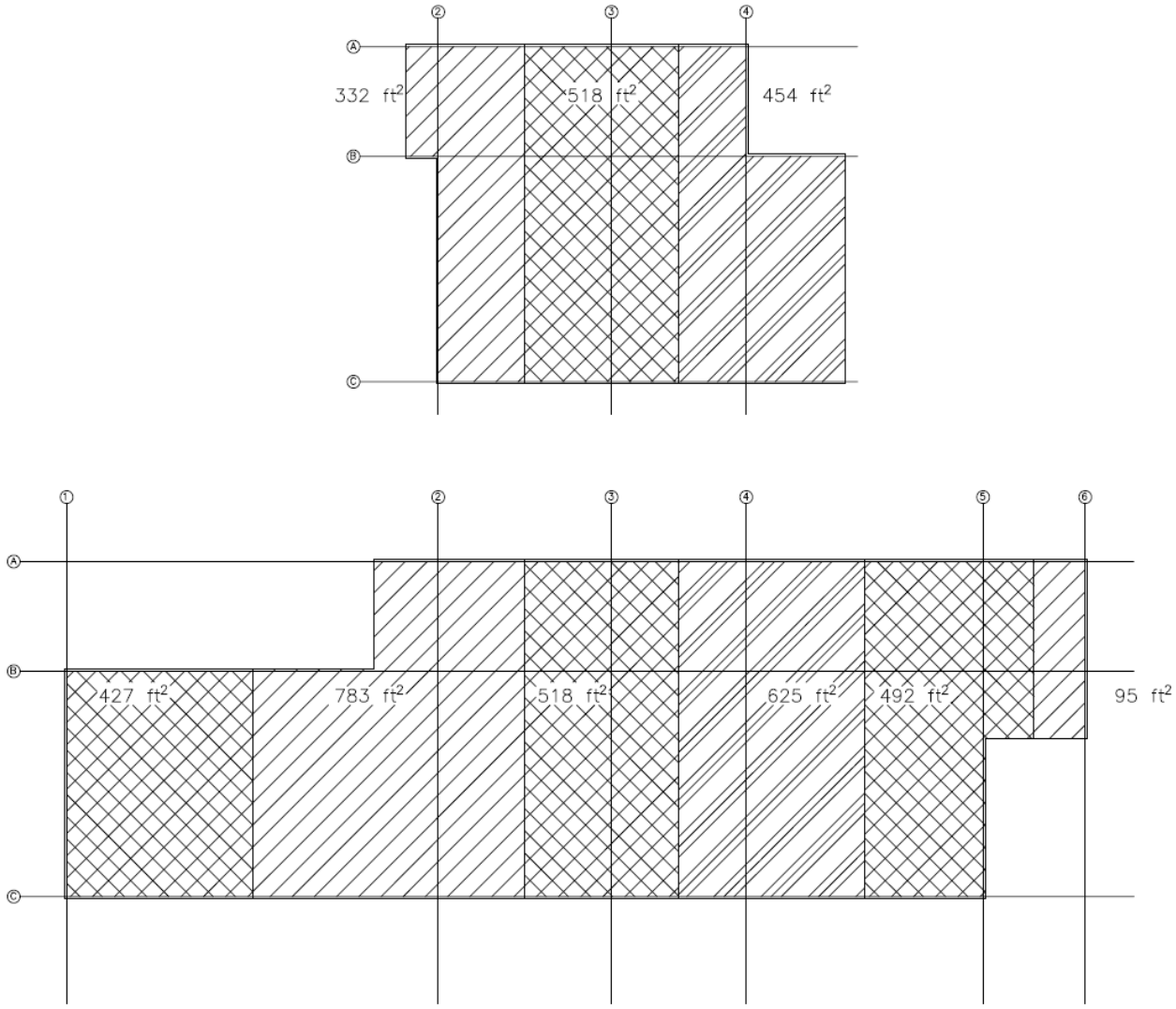


Figure B20. Index Building 2 tributary area.

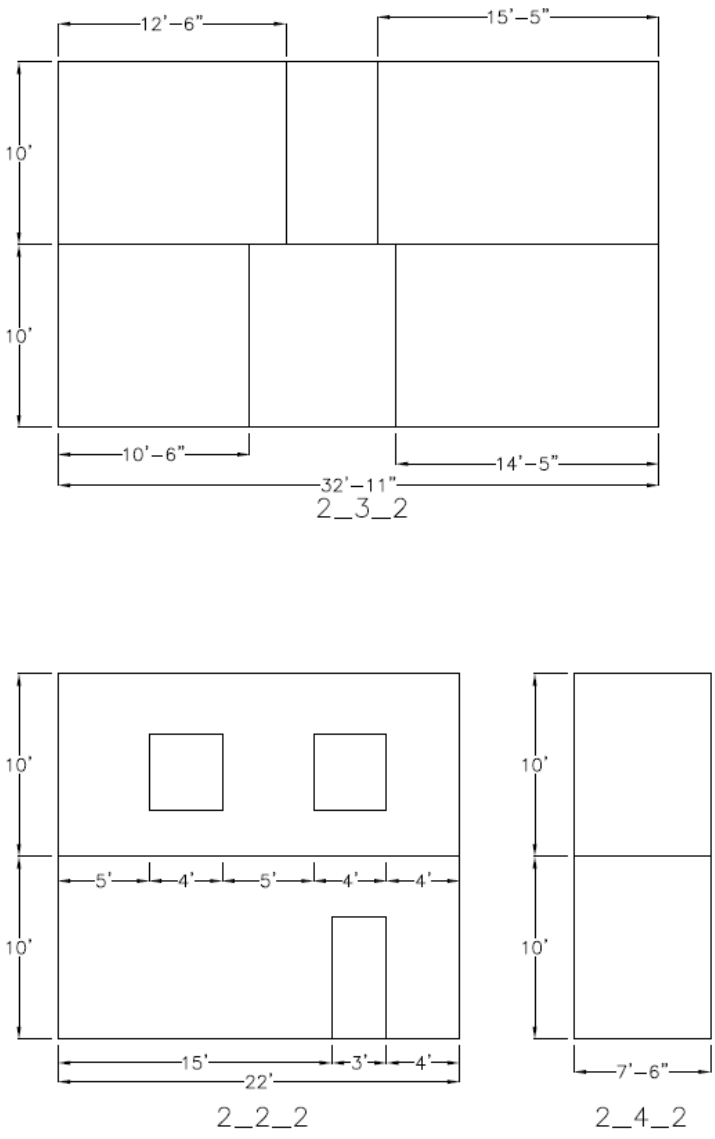


Figure B21. Index Building 2 extracted shear wall lines.

Table B16—Extracted archetypes, Index Building 2

Group No.		Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-1	2.5ft-20ft wall	Low aspect ratio panels	High	SDC D_{max}	2_4_2_1_LR_HG_DX_SP	2
PG-5			Low		2_4_2_1_LR_LG_DX_SP	14
PG-9		High aspect ratio panels	High		2_4_2_1_HR_HG_DX_SP	26
PG-13			Low		2_4_2_1_HR_LG_DX_SP	38
PG-25	20ft-60ft wall	Low aspect ratio panels	High		2_2_2_2_ LR_HG_ DX_SP	73
PG-29			Low		2_2_2_2_ LR_LG_ DX_SP	85
PG-41		Mixed aspect ratio	High		2_2_2_2_ MR_HG_ DX_SP	121
PG-45			Low		2_2_2_2_ MR_LG_ DX_SP	133

Table B17—Seismic weight detailed calculation, Index Building 2, 2-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			1250					36583.3			68964	36583.3			68964
	2	10		153.5	1295.7	86.25	813		43691	21071			43691	21071	
1			2953					92232.0			175597	145718.2			203168
	1	10		269.5	2151	119.50	1135		72534	29434			20703	29434	
Ground			0												

Table B18—Seismic weight summary, Index Building 2, 2-story

Level	h (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		69.0	69.0
	10		
1		175.6	203.2
	10		
Ground			

INPUT DATA

Total Height	h_n=	20.0	ft					
Total Weight	W=	245	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	3						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.3333						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	3,136						

DESIGN SUMMARY

Total base shear

V = 81.53

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	69	1,380	0.440	35.9		
	10.00						35.9	
1		10.0	176	1,756	0.560	45.7		359
	10.00						81.5	
		0.0						1,174

Seismic base shear calculation, low gravity, Index Building 2, 2-story, R = 3

INPUT DATA

Total Height	h_n=	20.0	ft					
Total Weight	W=	272	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	3						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.3333						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	3,412						

DESIGN SUMMARY

Total base shear

V = 90.73

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	69	1,380	0.404	36.7		
	10.00						36.7	
1		10.0	203	2,032	0.596	54.0		367
	10.00						90.7	
		0.0						1,274

Seismic base shear calculation, high gravity, Index Building 2, 2-story, R = 3

Table B19—Tributary load calculation, Index Building 2, 2-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 2	**2**	**332.00**	**0.255**	35.9	**9.13**	36.7	**9.34**
	3	518.00	0.397				
	4	**454.00**	**0.348**		**12.49**		**12.78**
Story 1	1	427.00	0.145	81.53	11.84	90.70	13.17
	2	**783.00**	**0.266**		**21.71**		**24.16**
	3	518.00	0.176		14.37		15.98
	4	**625.00**	**0.213**		**17.33**		**19.28**
	5	492.00	0.167		13.64		15.18
	6	95.00	0.032		2.63		2.93

Table B20—Archetype 2 (2_4_2_1_LR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	5	D	2605.0	2	1	0.79	1.79	2555.5	13.03	12.78	1.02
1	1	S	5	6.875	5	A3	8	D	4168.0	2	1 3/4	0.85	2.51	3856.3	20.84	19.28	1.08

*Values shown are for unreinforced option and before applying C_d.

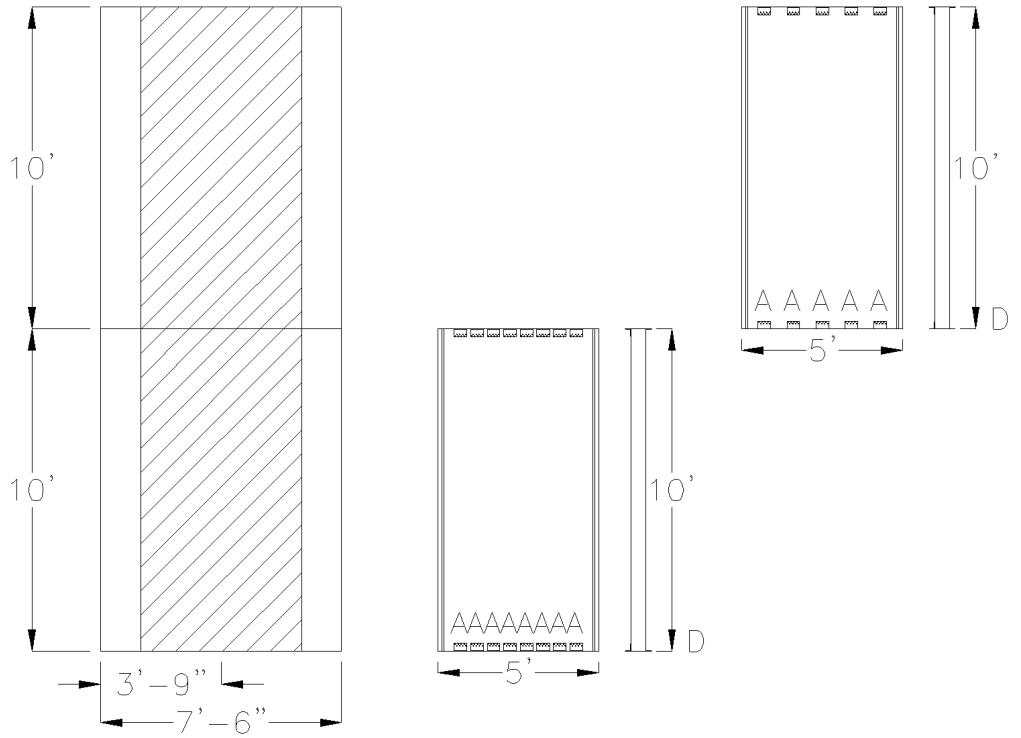


Figure B22. Archetype 2.

Table B21—Archetype 14 (2_4_2_1_LR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	5	D	2605.0	2	1	0.78	1.78	2498.3	13.03	12.49	1.04
1	1	S	5	6.875	5	A3	7	D	3647.0	2	1 3/4	0.80	2.40	3466.6	18.24	17.33	1.05

*Values shown are for unreinforced option and before applying C_d.

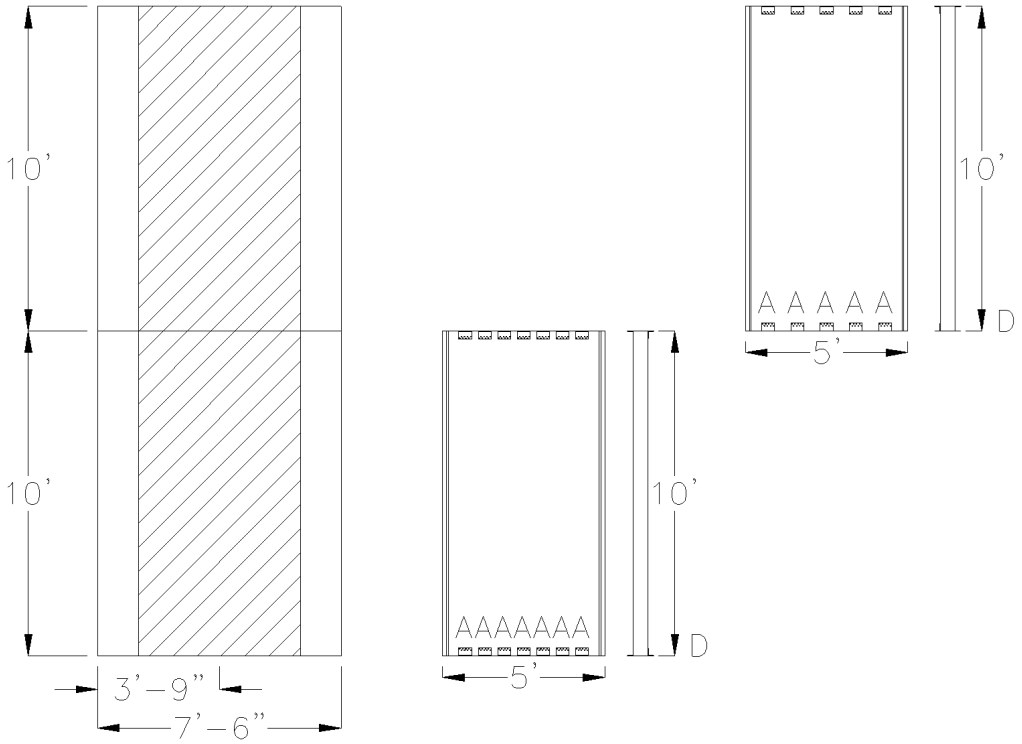


Figure B23. Archetype 14.

Table B22—Archetype 26 (2_4_2_1_HR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.82	1.72	1703.7	15.63	12.78	1.22
1	3	M	2.5	6.875	5	A3	3	D	3126.0	2	1 3/4	0.83	2.58	2570.9	23.45	19.28	1.22

*Values shown are for unreinforced option and before applying C_d.

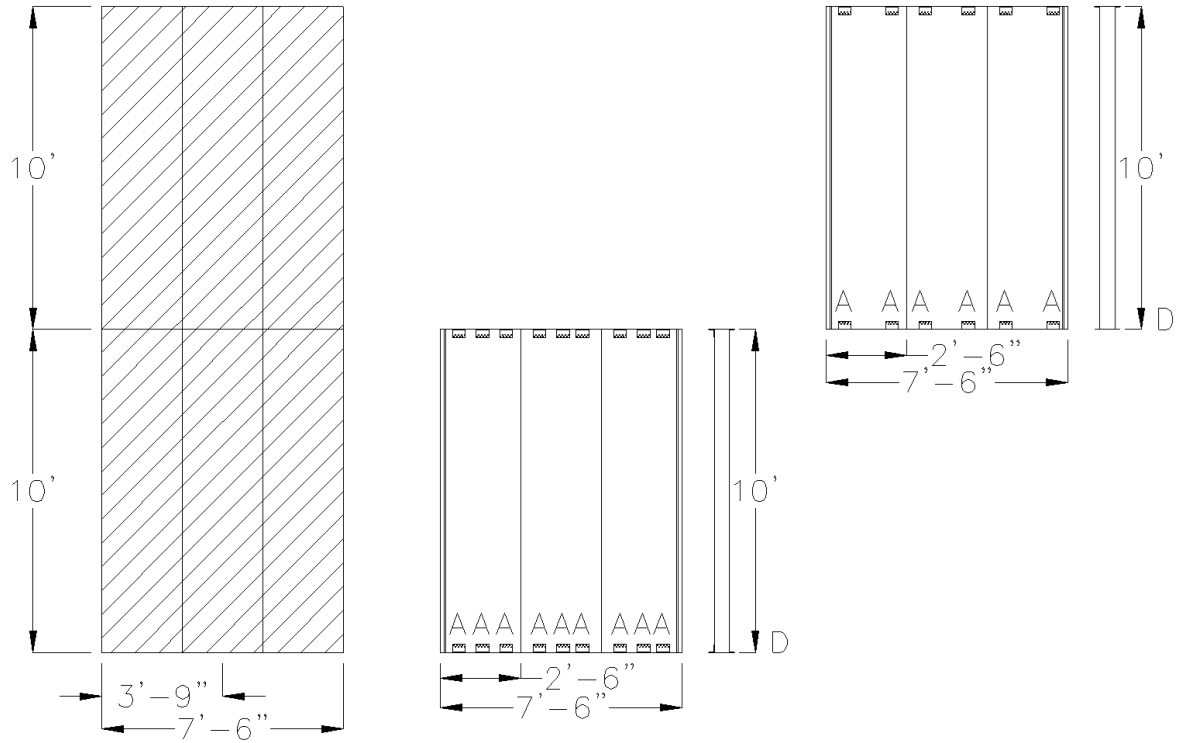


Figure B24. Archetype 26.

Table B23—Archetype 38 (2_4_2_1_HR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.81	1.71	1665.5	15.63	12.49	1.25
1	3	M	2.5	6.875	5	A3	3	D	3126.0	2	1 1/2	0.79	2.45	2311.0	23.45	17.33	1.35

*Values shown are for unreinforced option and before applying C_d.

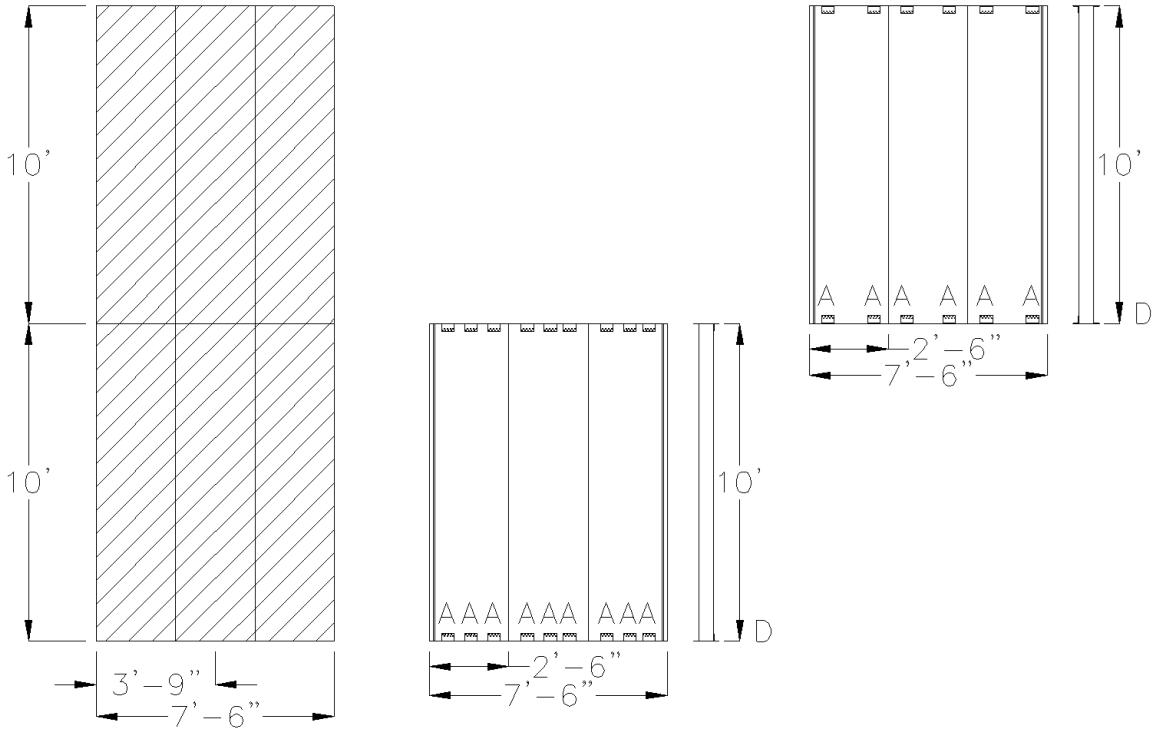


Figure B25. Archetype 38.

Table B24—Archetype 73 (2_2_2_2_ LR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.63	0.83	934.4	-	-	-
1	1	S	5	4.125	3	A3	5	D	2605.0	2	1 1/4	0.81	1.67	2415.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.63	0.83	934.4	10.42	9.34	1.12
1	1	S	5	4.125	3	A3	5	D	2605.0	2	1 1/4	0.81	1.67	2415.6	26.05	24.16	1.08

*Values shown are for unreinforced option and before applying C_d.

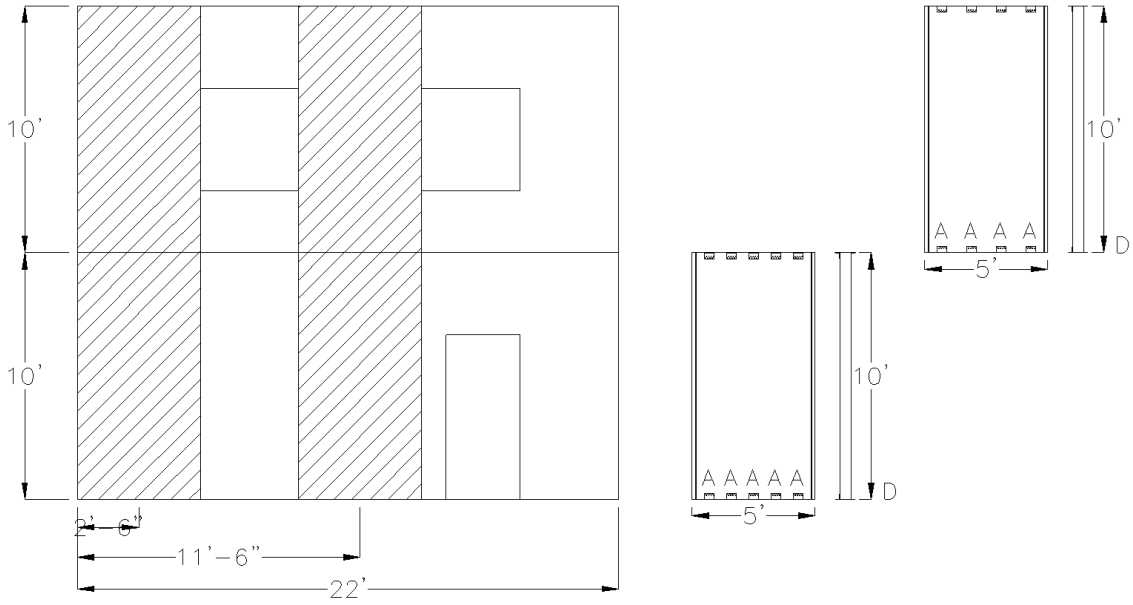


Figure B26. Archetype 73.

Table B25—Archetype 85 (2_2_2_2_ LR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.62	0.82	913.5	-	-	-
1	1	S	5	4.125	3	A3	5	D	2605.0	2	1 1/4	0.75	1.61	2171.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.62	0.82	913.5	10.42	9.13	1.14
1	1	S	5	4.125	3	A3	5	D	2605.0	2	1 1/4	0.75	1.61	2171.4	26.05	21.71	1.20

*Values shown are for unreinforced option and before applying C_d.

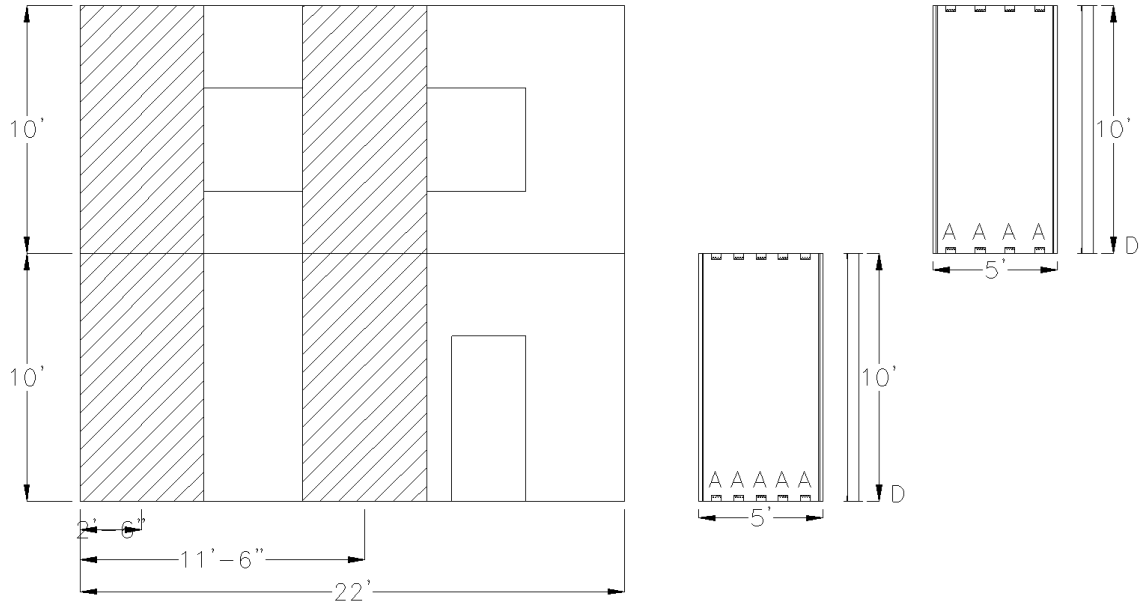


Figure B27. Archetype 85.

Table B26—Archetype 121 (2_2_2_2_ MR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.73	0.46	607.7	-	-	-
1	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/8	1.01	0.89	1619.0	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	5	S	1302.5	2	5/8	0.73	0.96	1261.1	11.72	9.34	1.25
1	1	S	5	4.125	3	A3	7	D	3647.0	2	1 1/4	1.01	1.77	3212.2	28.66	24.16	1.19

*Values shown are for unreinforced option and before applying C_d.

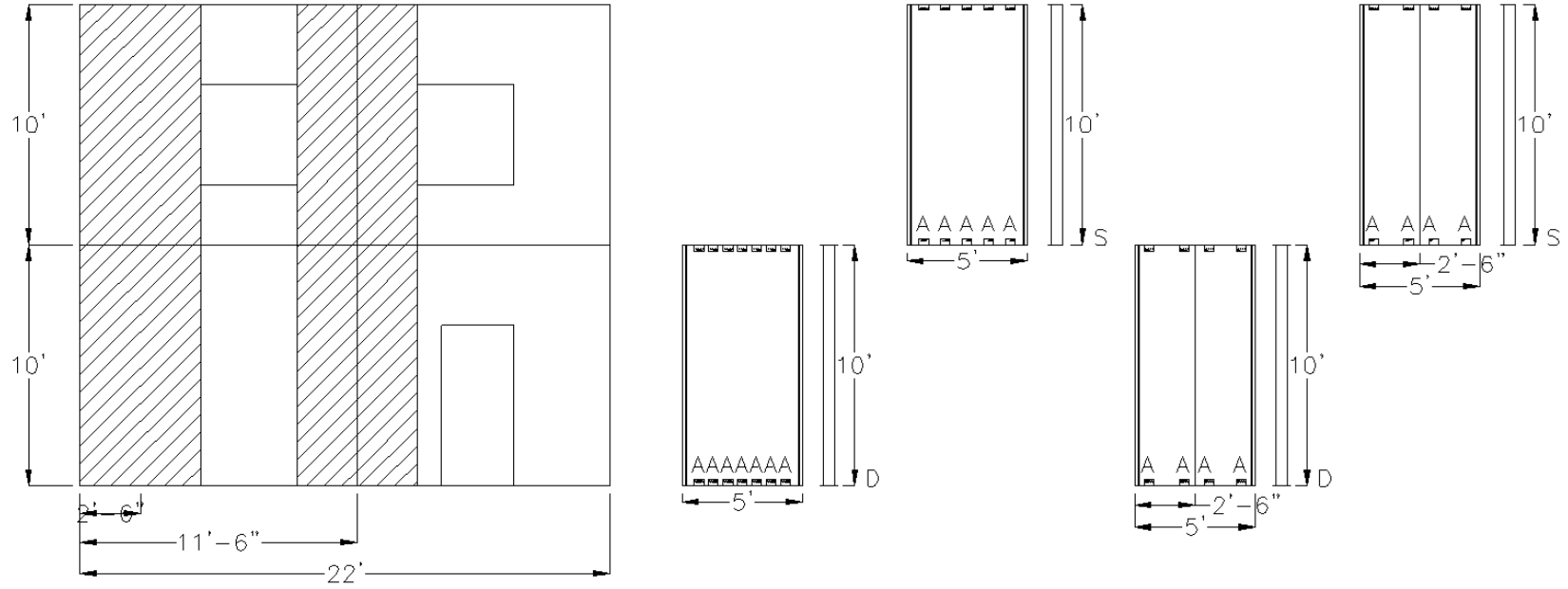


Figure B28. Archetype 121.

Table B27—Archetype 133 (2_2_2_2_ MR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.79	0.50	700.6	-	-	-
1	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/4	0.97	0.89	1551.6	-	-	-
Story	No. of Panels	Configuration	Panel Length (ft)			Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	5	S	1302.5	2	1/2	0.79	0.80	1126.4	11.72	9.13	1.28
1	1	S	5	4.125	3	A3	6	D	3126.0	2	1 1/8	0.97	1.60	2791.3	26.05	21.71	1.20

*Values shown are for unreinforced option and before applying C_d.

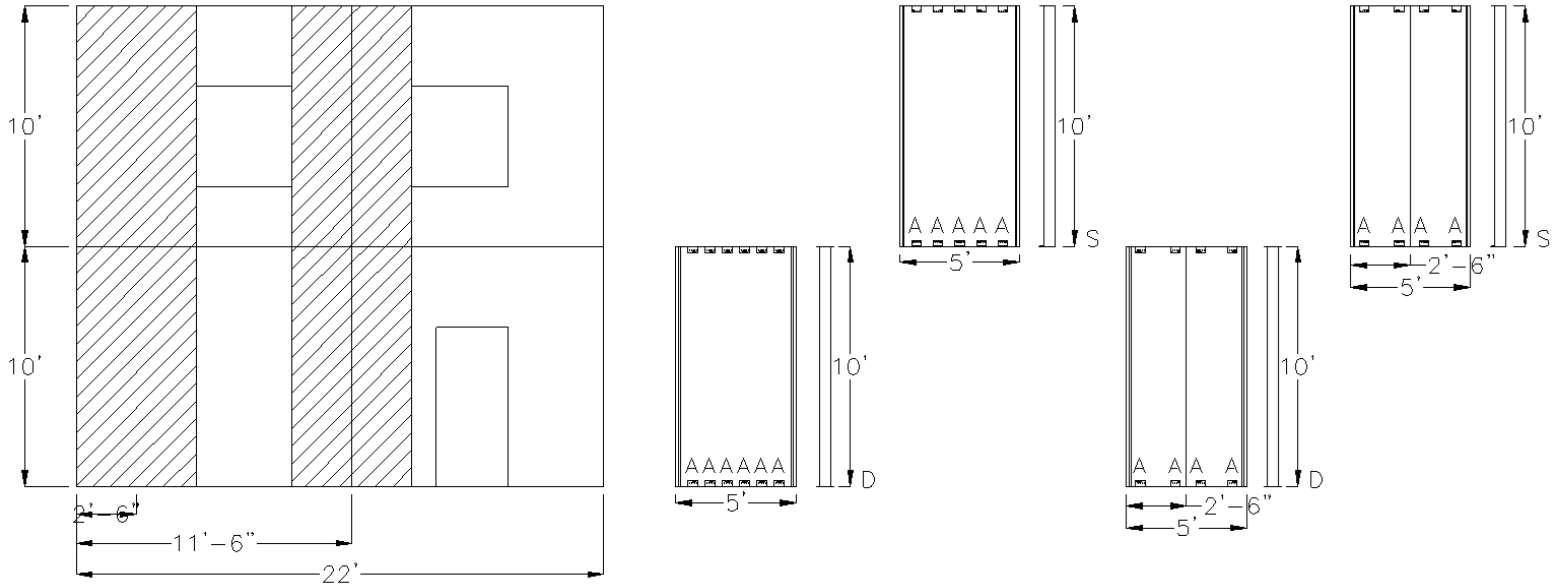


Figure B29. Archetype 133.

INPUT DATA

Total Height	h_n=	**20.0**	ft				
Total Weight	W=	**245**	k				
Seismic Design Category		**Dmax**					
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)				
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**	
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**	
	S_{DS} =	**1.000**	g ,				
	S_{D1} =	**0.600**	g				
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**					
The coefficient(ASCE Tab. 12.2.1)	R =	**4**					
	x =	**0.75**	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)				
	Cu=	**1.40**					
	T=Cu*Ta=	0.2648					
	Ts=	0.6					
	Cs=	0.25					
	k =	**1.00**	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	3,136					

DESIGN SUMMARY

Total base shear

V = 61.15

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	**69**	1,380	0.440	26.9		
	10.00						26.9	
1		10.0	**176**	1,756	0.560	34.2		269
	10.00						61.2	
		0.0						881

Seismic base shear calculation, low gravity, Index Building 2, 2-story, R = 4

INPUT DATA

Total Height	h_n=	**20.0**	ft				
Total Weight	W=	**272**	k				
Seismic Design Category		**Dmax**					
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)				
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**	
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**	
	S_{DS} =	**1.000**	g ,				
	S_{D1} =	**0.600**	g				
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**					
The coefficient(ASCE Tab. 12.2.1)	R =	**4**					
	x =	**0.75**	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)				
	Cu=	**1.40**					
	T=Cu*Ta=	0.2648					
	Ts=	0.6					
	Cs=	0.25					
	k =	**1.00**	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	3,412					

DESIGN SUMMARY

Total base shear

V = 68.05

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	**69**	1,380	0.404	27.5		
	10.00						27.5	
1		10.0	**203**	2,032	0.596	40.5		275
	10.00						68.1	
		0.0						956

Seismic base shear calculation, high gravity, Index Building 2, 2-story, R = 4

Table B28—Tributary load calculation, Index Building 2, 2-story, N-S direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 2	**2**	**332.00**	**0.255**	26.9	**6.85**	27.5	**7.00**
	3	518.00	0.397				
	4	**454.00**	**0.348**		**9.37**		**9.57**
Story 1	1	427.00	0.145	61.15		68.10	
	2	**783.00**	**0.266**		**16.29**		**18.14**
	3	518.00	0.176				
	4	**625.00**	**0.213**		**13.00**		**14.48**
	5	492.00	0.167				
	6	95.00	0.032				

Table B29—Archetype 26 (2_4_2_1_HR_HG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	7/8	0.74	1.43	1276.6	11.72	9.57	1.22
1	3	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/4	0.79	2.03	1930.3	15.63	14.48	1.08

*Values shown are for unreinforced option and before applying C_d.

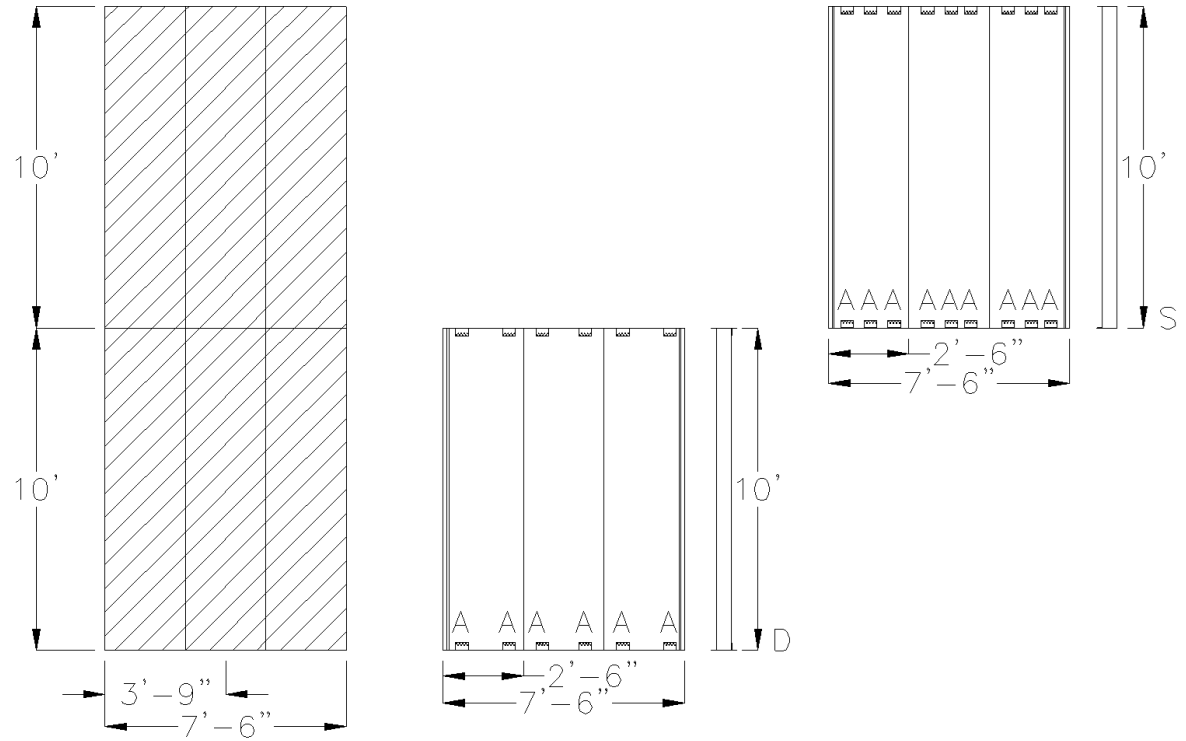


Figure B30. Archetype 26.

Table B30—Archetype 38 (2_4_2_1_HR_LG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	7/8	0.73	1.42	1249.2	11.72	9.37	1.25
1	3	M	2.5	6.875	5	A3	4	S	2084.0	2	1 1/4	0.73	1.98	1733.3	15.63	13.00	1.20

*Values shown are for unreinforced option and before applying C_d.

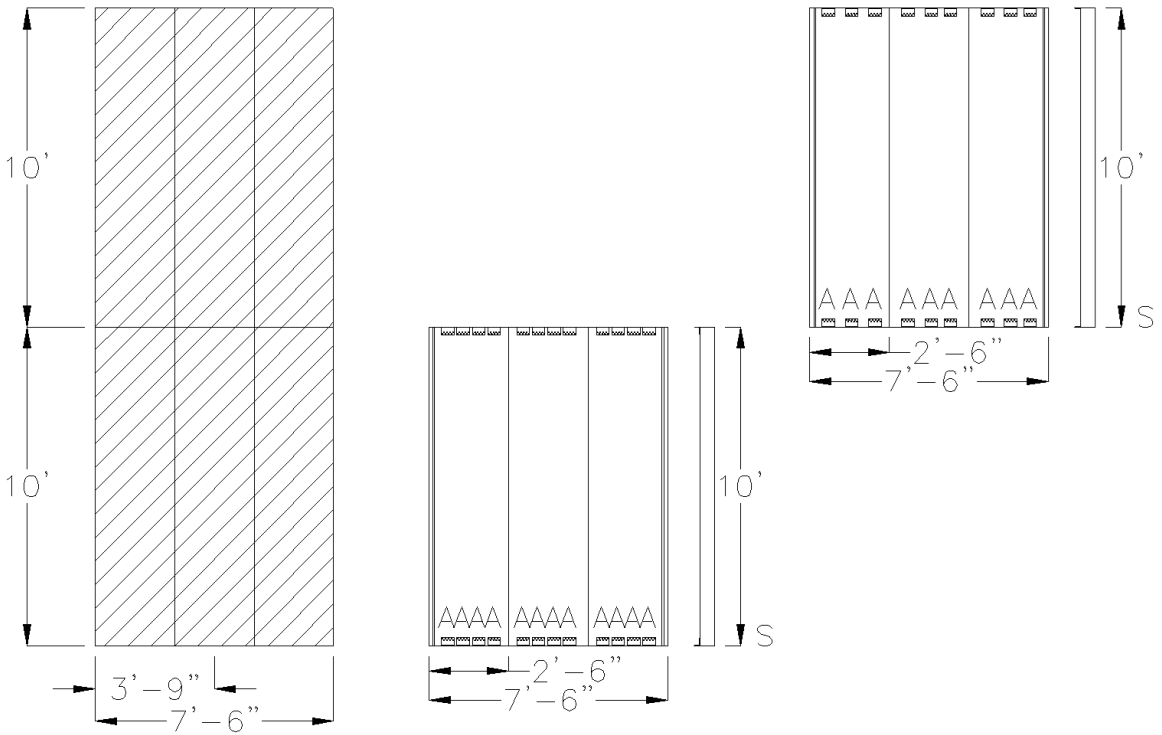


Figure B31. Archetype 38.

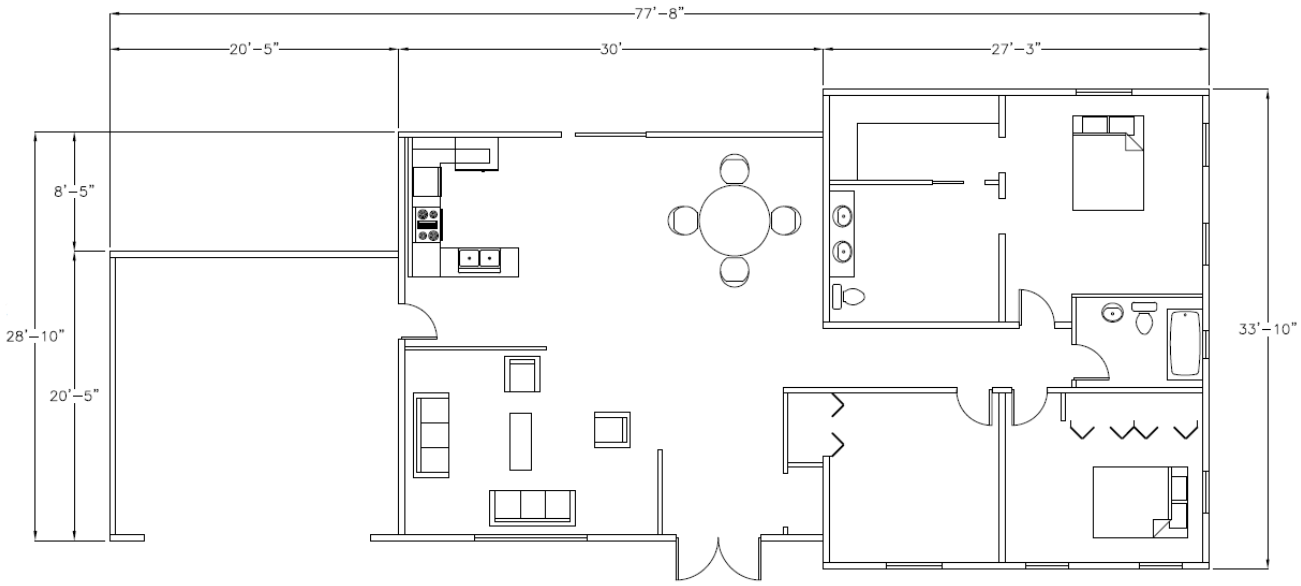


Figure B32. Index Building 3 floor plan.

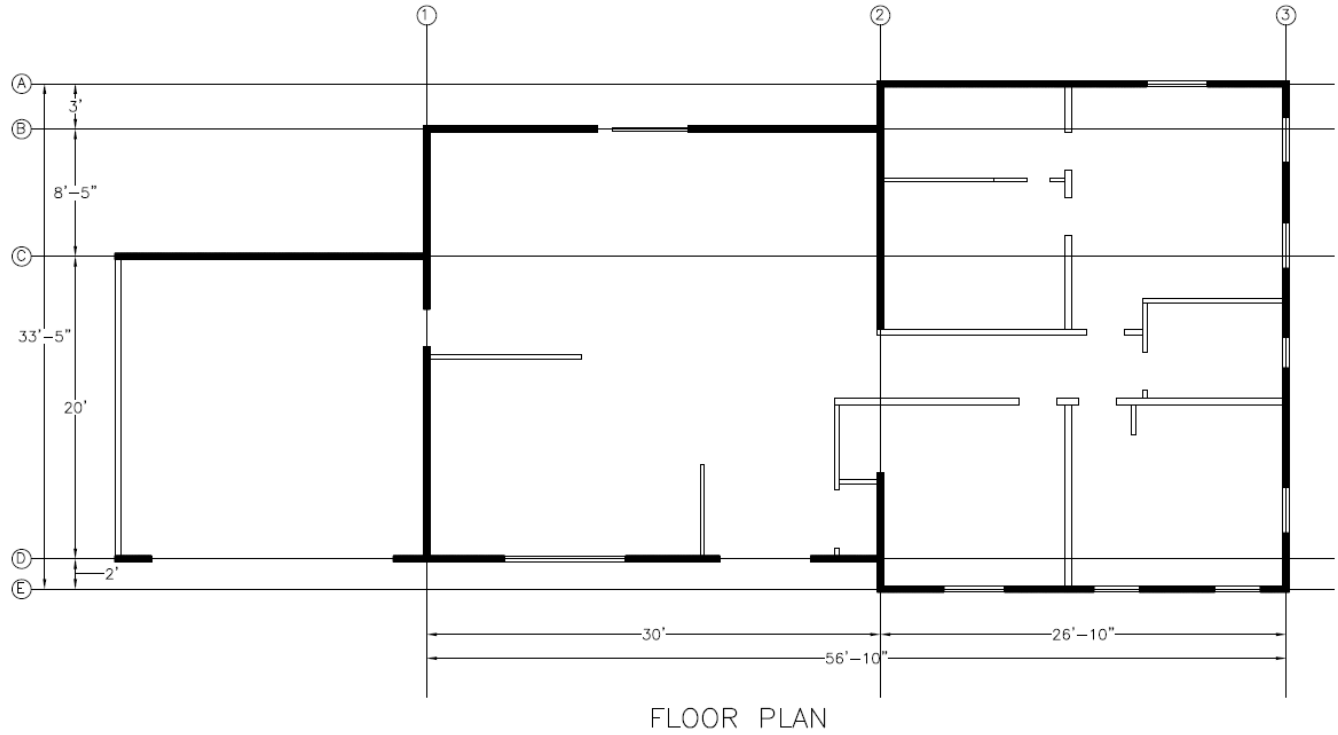


Figure B33. Index Building 3 assigned shear walls.

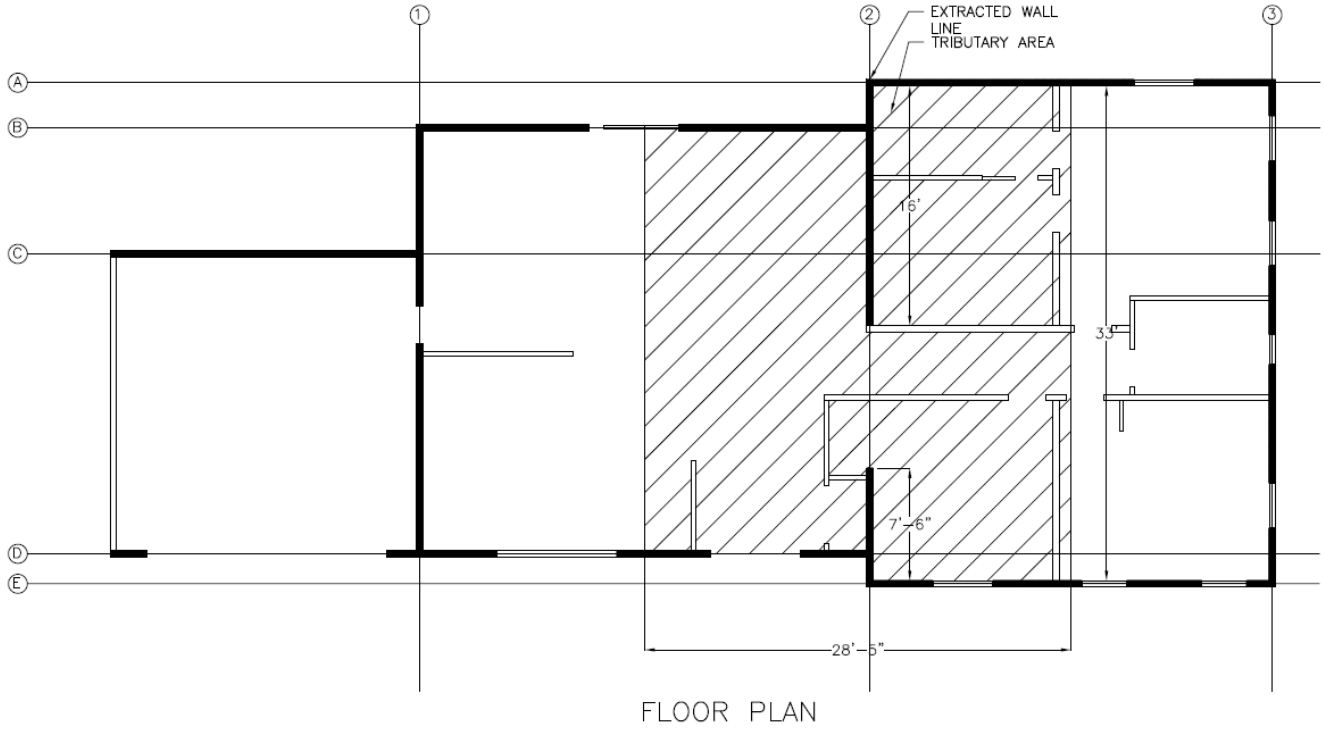


Figure B34. Index Building 3 tributary area.

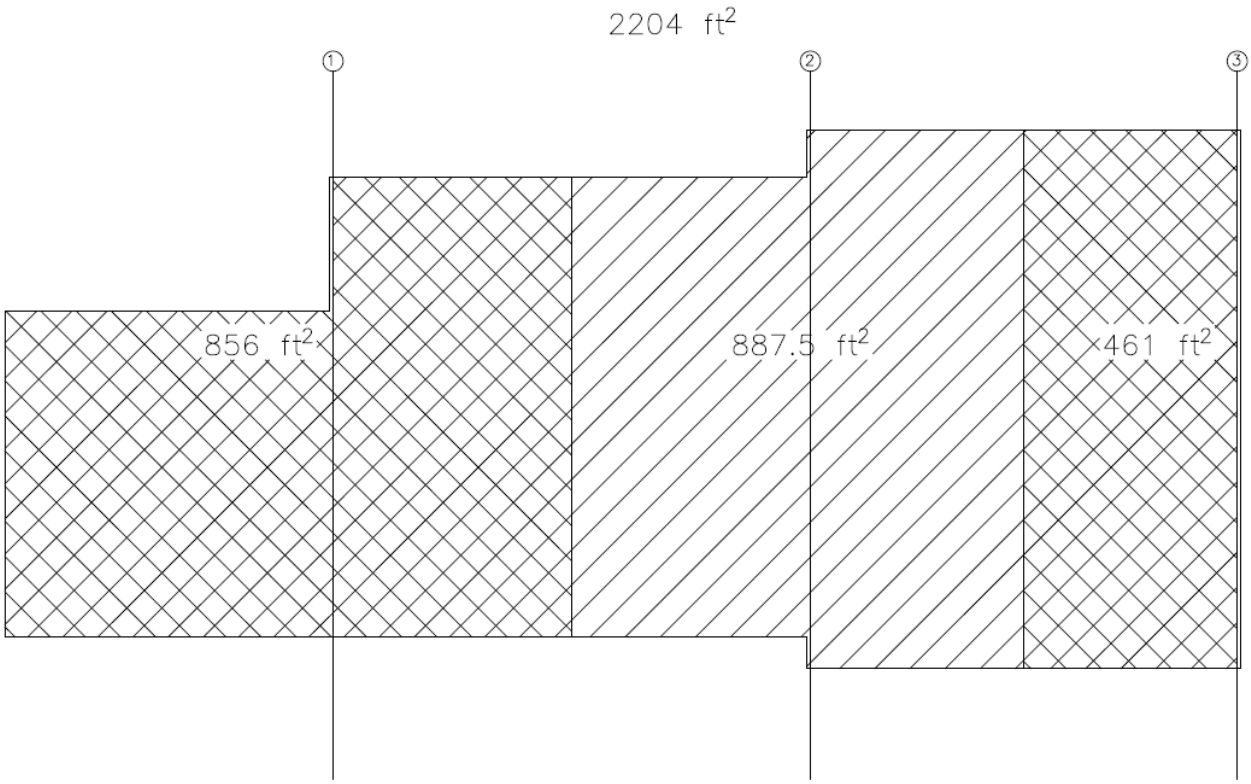


Figure B35. Index Building 3 tributary area.

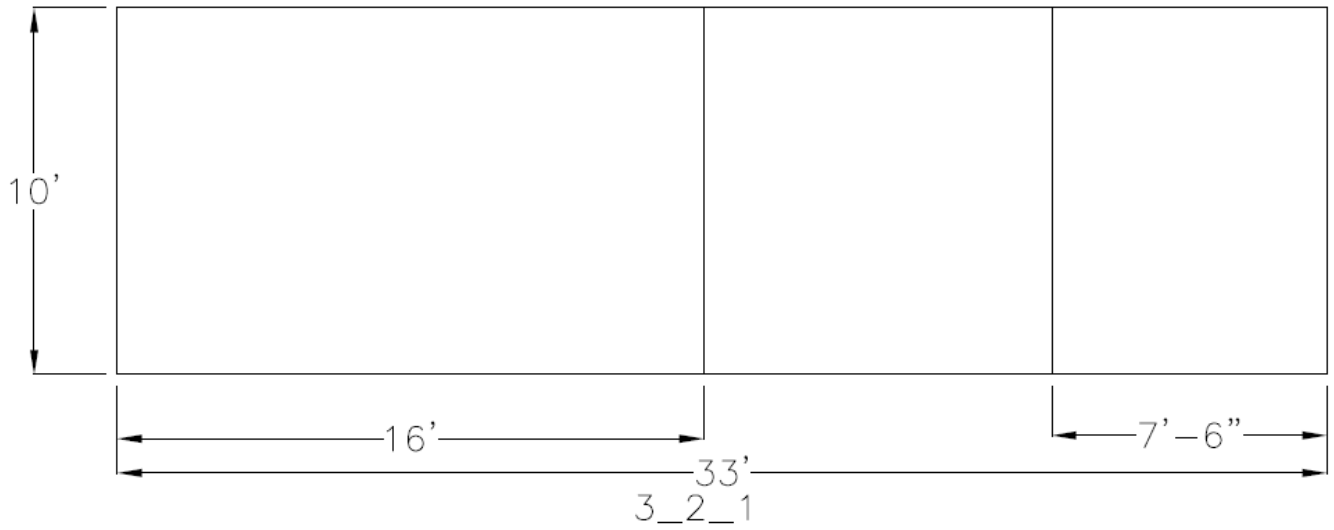


Figure B36. Index Building 3 extracted shear wall line.

Table B31—Extracted archetypes, Index Building 3

Group No.	Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-25	20ft-60ft wall, Low aspect ratio panels	High		3_2_1_2_ LR_HG_ DX_SP	74
PG-29	20ft-60ft wall, Low aspect ratio panels	Low		3_2_1_2_ LR_LG_ DX_SP	86
PG-41	20ft-60ft wall, Mixed aspect ratio	High		3_2_1_2_ MR_HG_ DX_SP	122
PG-45	20ft-60ft wall, Mixed aspect ratio	Low		3_2_1_2_ MR_LG_ DX_SP	134

Table B32—Seismic weight detailed calculation, Index Building 3, 1-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)
Roof			2204					64503.7			109899	64503.7			109899
	6	10		221.3	1898.7	113.21	1032		64025	26765			64025	26765	
Ground			0												

Table B33—Seismic weight summary, Index Building 3, 1-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
1		109.9	109.9
	10		
Ground			

INPUT DATA

Total Height	h_n=	10.0	ft			
Total Weight	W=	110	k			
Seismic Design Category		Dmax				
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)			
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500
	S_{DS} =	1.000	g ,			
	S_{D1} =	0.600	g			
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)			
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02				
The coefficient(ASCE Tab. 12.2.1)	R =	3				
	x =	0.75	, (ASCE Tab 12.8-2)			
	$T_a = C_t(h_n)^x$ =	0.11	Sec, (ASCE 12.8.2.1)			
	Cu=	1.40				
	T=Cu*Ta=	0.25				
	Ts=	0.6				
	Cs=	0.3333				
	k =	1.00	, (ASCE 12.8.3, pg 130)			
	$\Sigma w_x h^k$ =	1,099				

DESIGN SUMMARY

Total base shear

V = 36.63

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
1		10.0	110	1,099	1.000	36.6		
	10.00						36.6	
		0.0						366

Seismic base shear calculation, high gravity and low gravity, Index Building 3, 1-story, R = 3

Table B34—Tributary load calculation, Index Building 3, 1-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 1	1	856.00	0.388	36.6	14.73	36.6	14.73
	2	887.50	0.403				
	3	461.00	0.209				

Table B35—Archetype 74 (3_2_1_2_ LR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	4.125	3	A3	4	S	1042.0	2	5/8	0.33	4.46	892.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	1	S	5	4.125	3	A3	2	S	521.0	2	3/8	0.33	0.45	269.8	18.24	14.73	1.24

*Values shown are for unreinforced option and before applying C_d.

Table B36—Archetype 86 (3_2_1_2_ LR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	4.125	3	A3	4	S	1042.0	2	5/8	0.33	4.46	892.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	1	S	5	4.125	3	A3	2	S	521.0	2	3/8	0.33	0.45	269.8	18.24	14.73	1.24

*Values shown are for unreinforced option and before applying C_d.

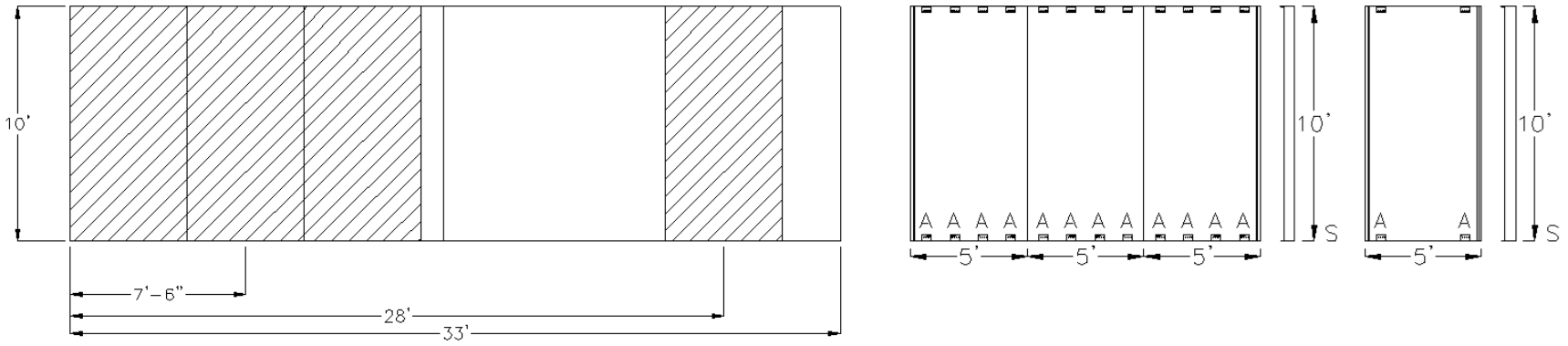


Figure B37. Archetype 74, Archetype 86.

Table B37—Archetype 122 (3_2_1_2_ MR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.33	0.51	201.8	-	-	-
Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	4.125	3	A3	4	S	1042.0	2	5/8	0.33	4.45	881.4	23.45	14.73	1.59

*Values shown are for unreinforced option and before applying C_d.

Table B38—Archetype 134 (3_2_1_2_ MR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.33	0.51	201.8	-	-	-
Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	4.125	3	A3	4	S	1042.0	2	5/8	0.33	4.45	881.4	23.45	14.73	1.59

*Values shown are for unreinforced option and before applying C_d.

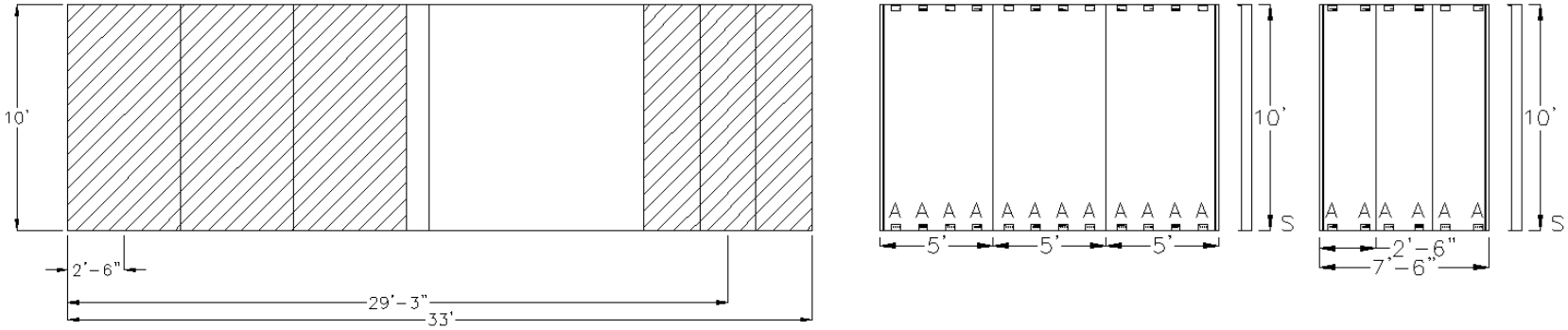


Figure B38. Archetype 122, Archetype 134.

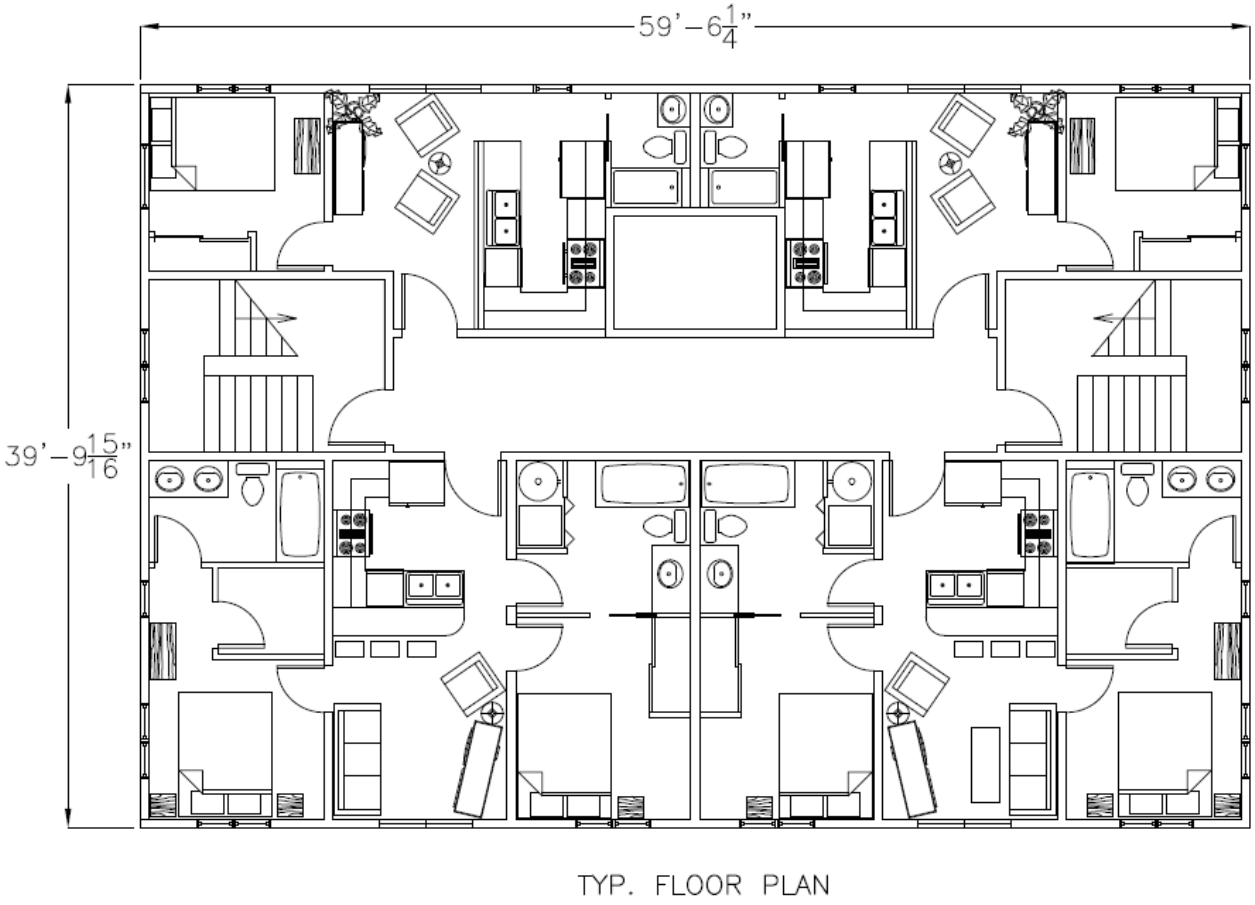


Figure B39. Index Building 4 floor plan.

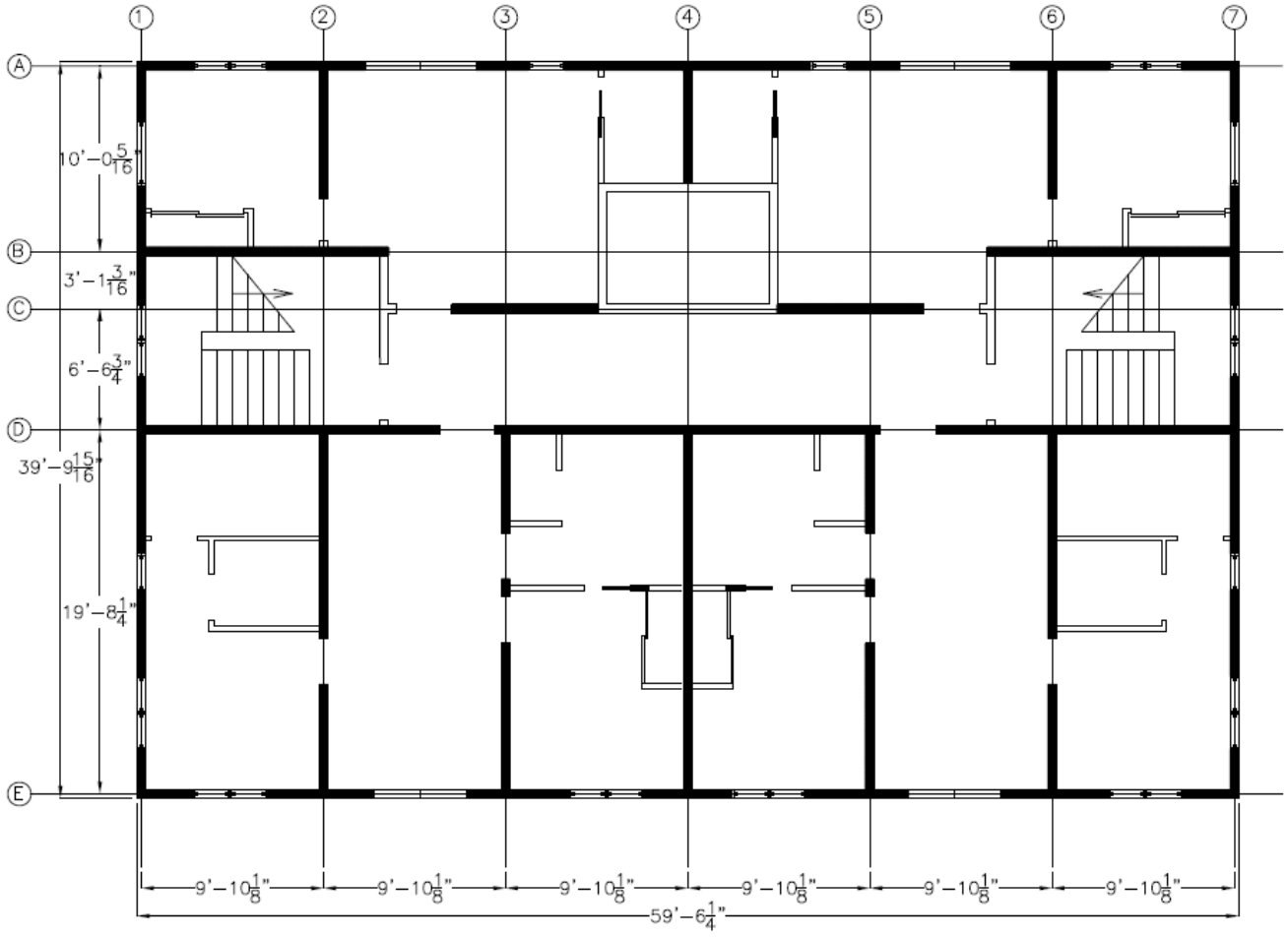


Figure B40. Index Building 4 assigned shear walls.

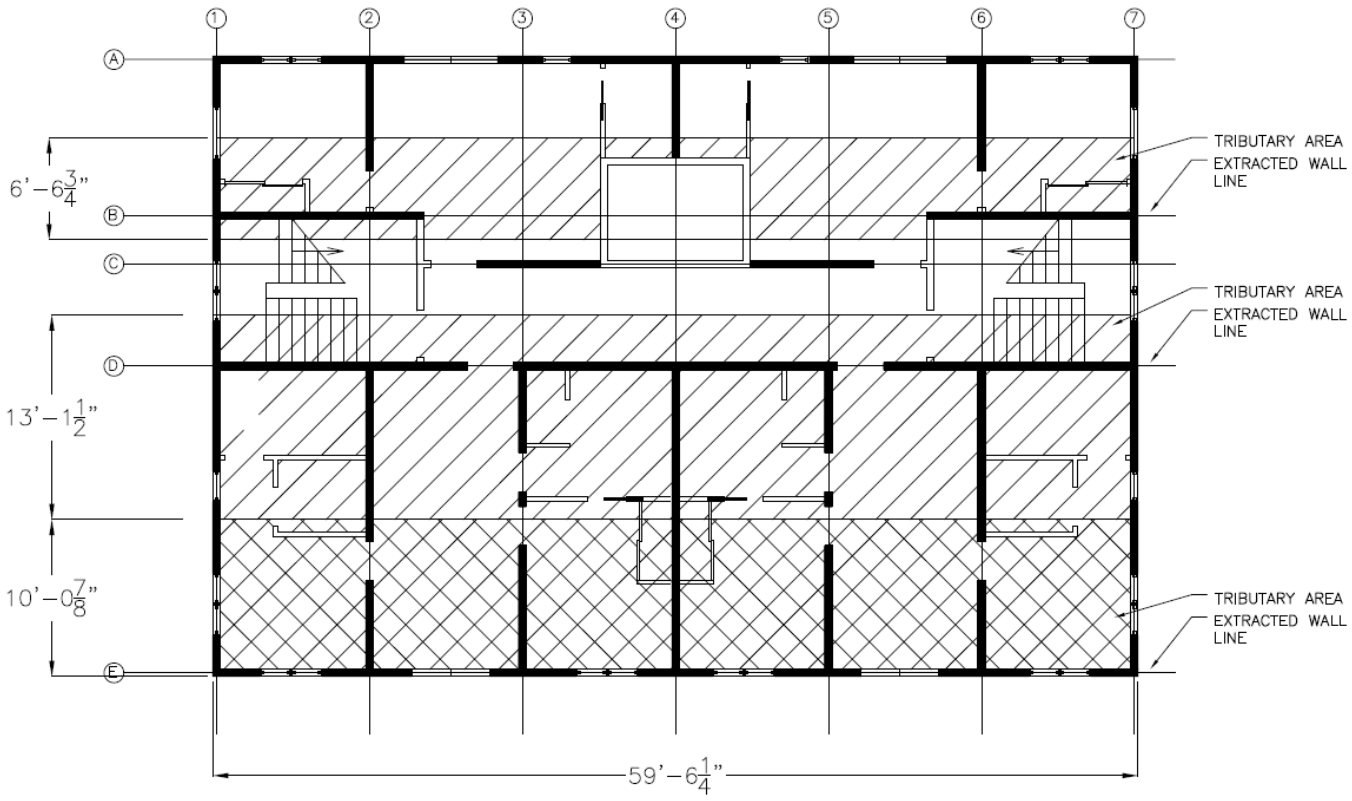


Figure B41. Index Building 4 tributary area.

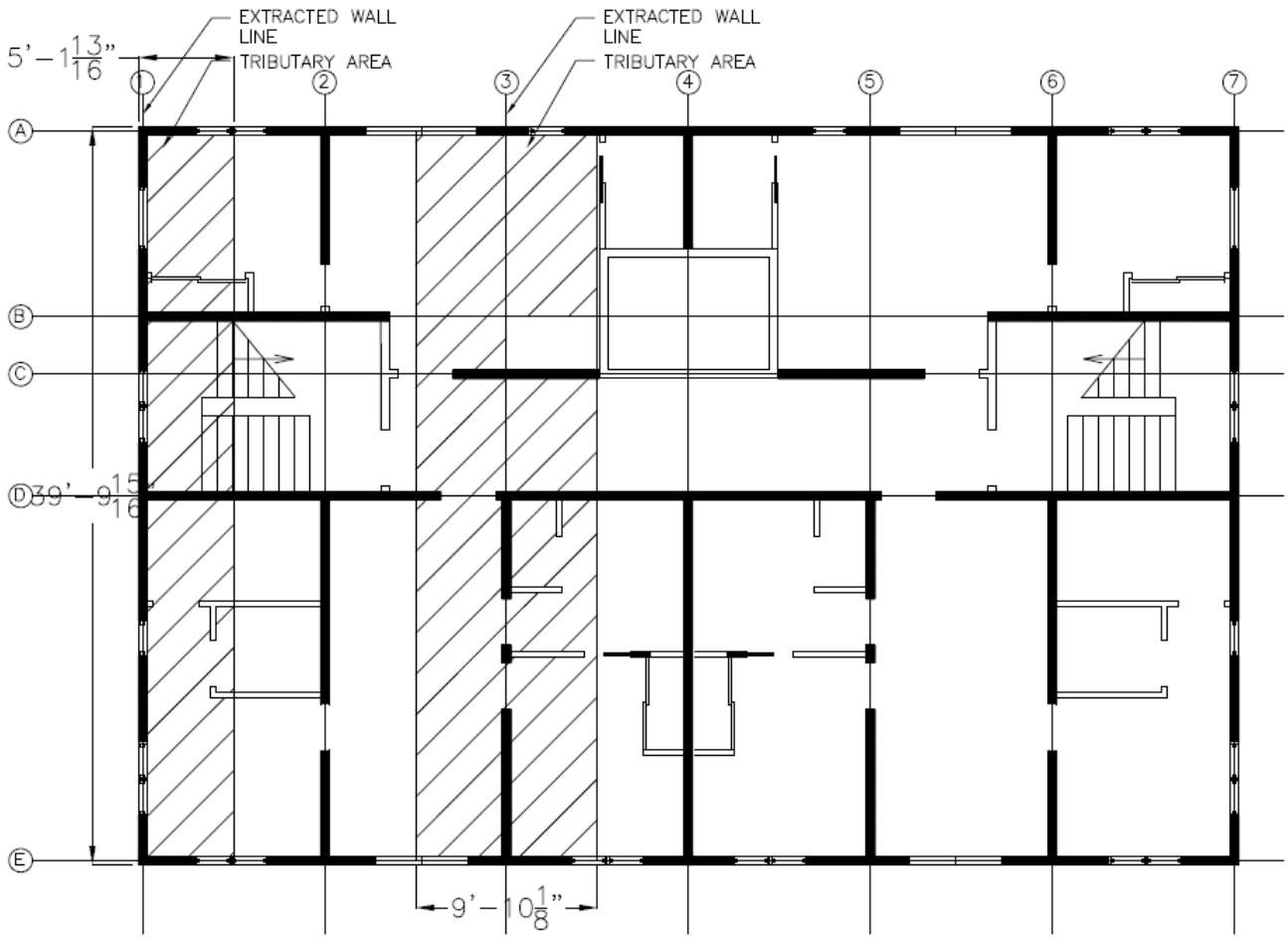


Figure B42. Index Building 4 tributary area.

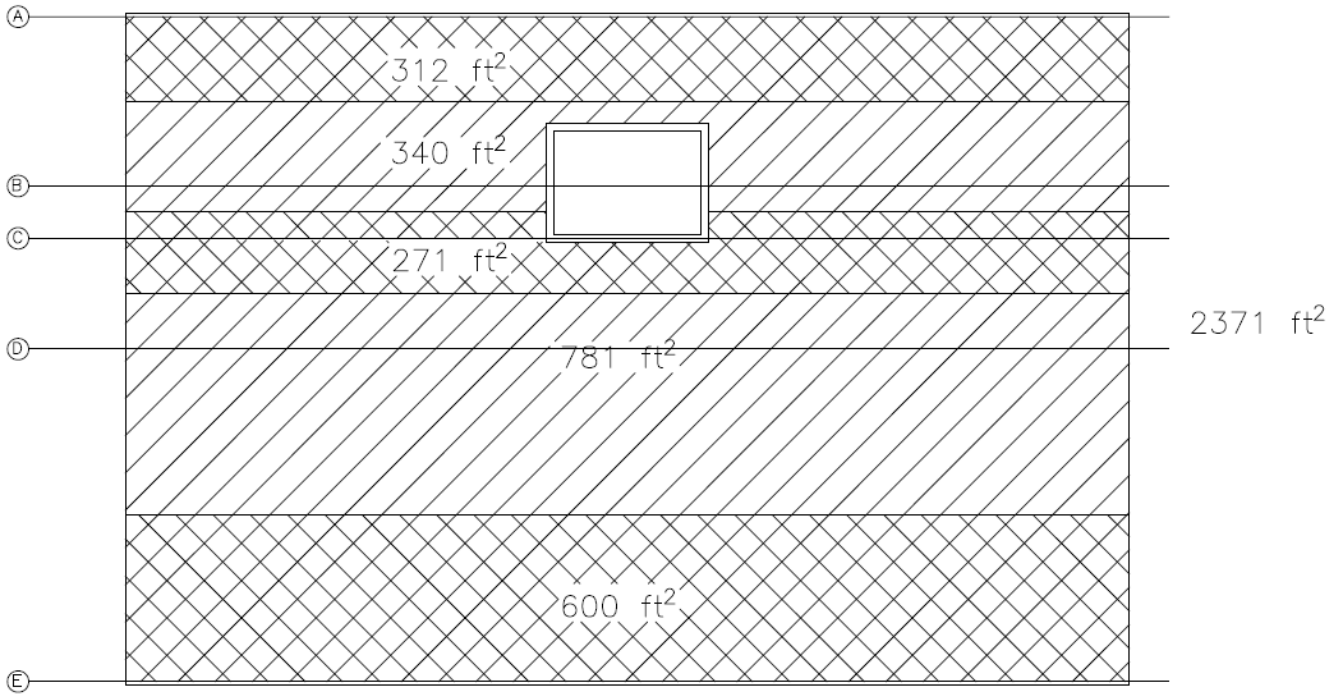


Figure B43. Index Building 4 tributary area.

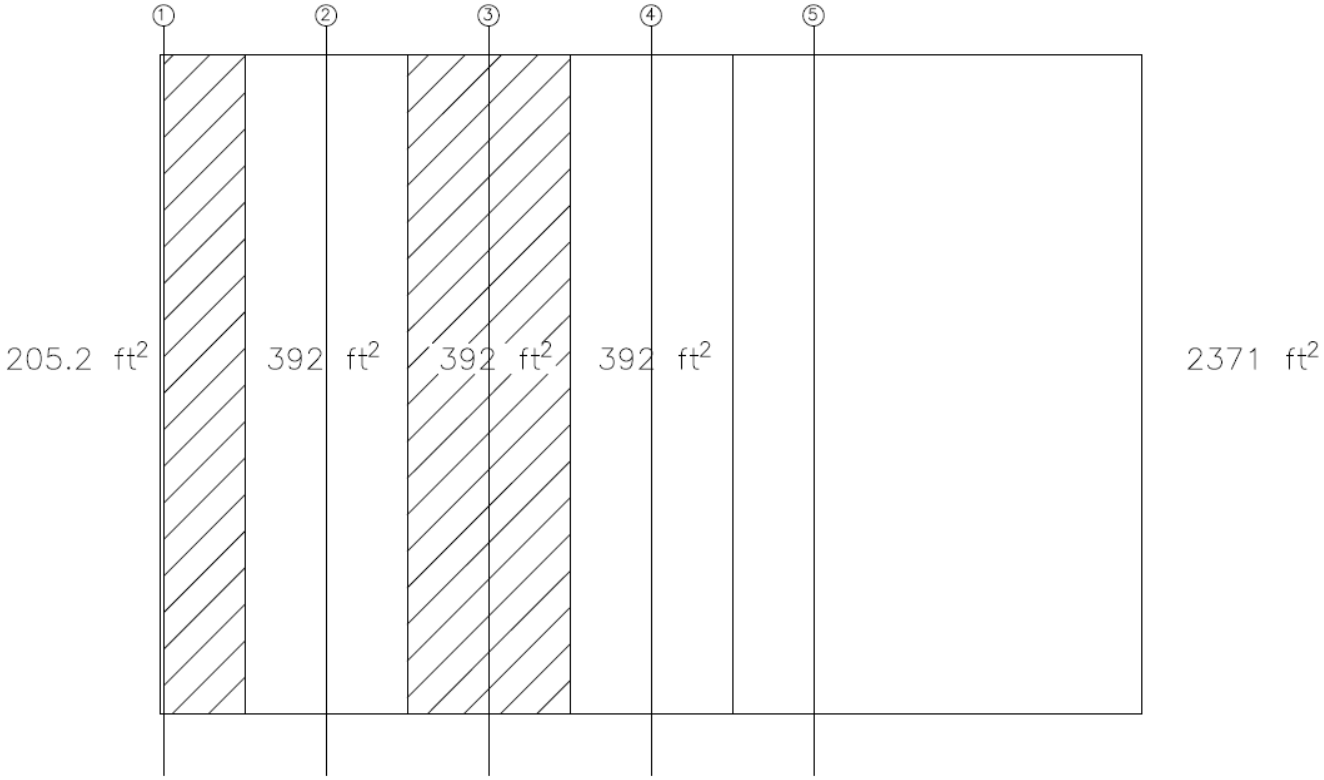


Figure B44. Index Building 4 tributary area.

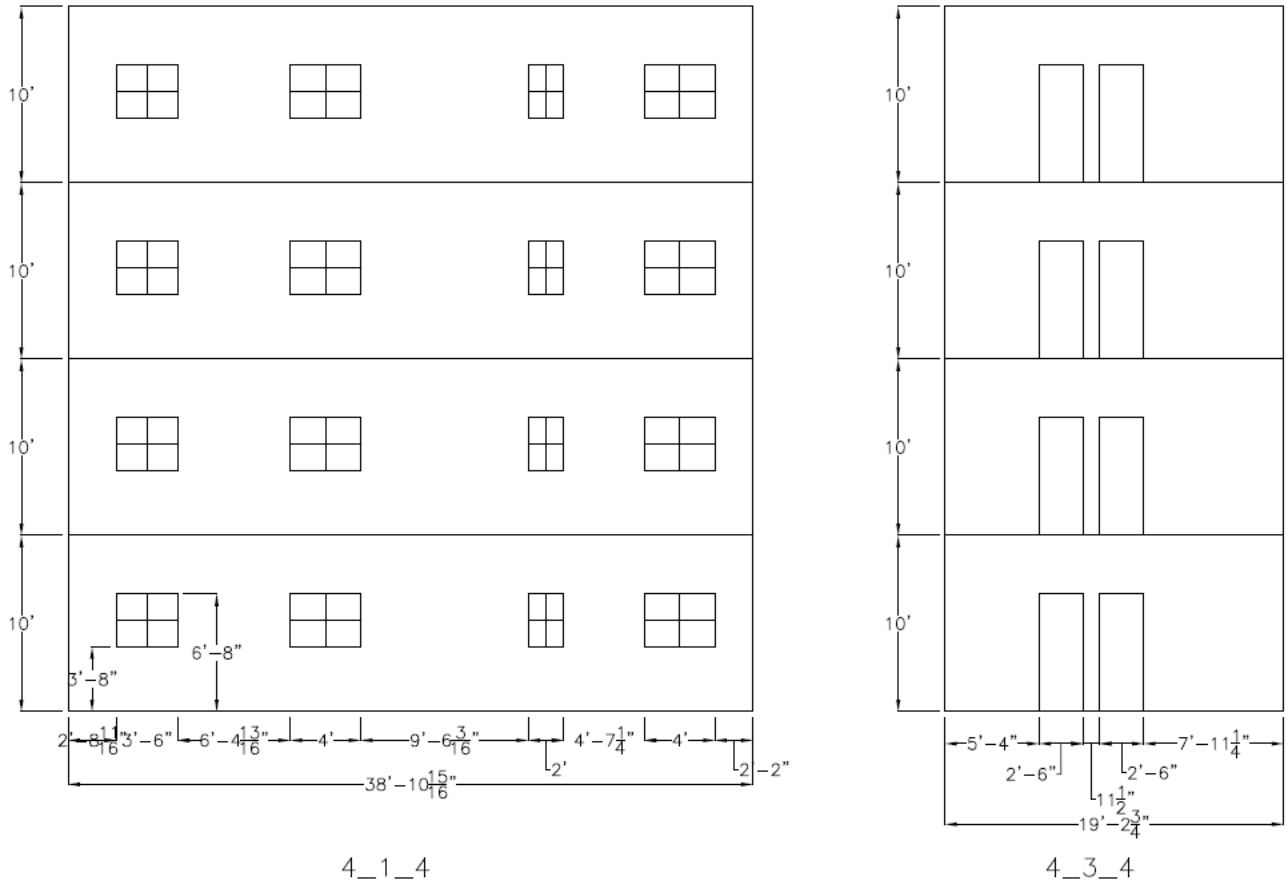


Figure B45. Index Building 4 extracted shear wall lines.

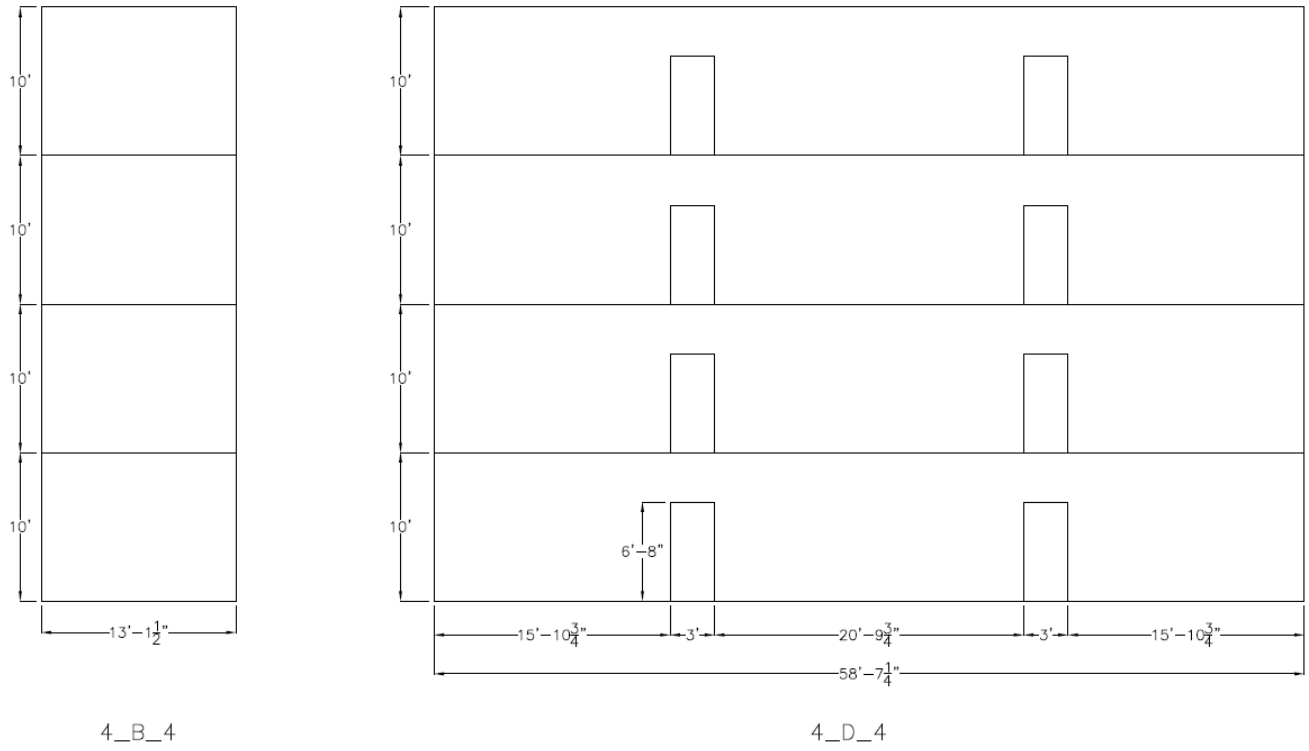


Figure B46. Index Building 4 extracted shear wall lines.

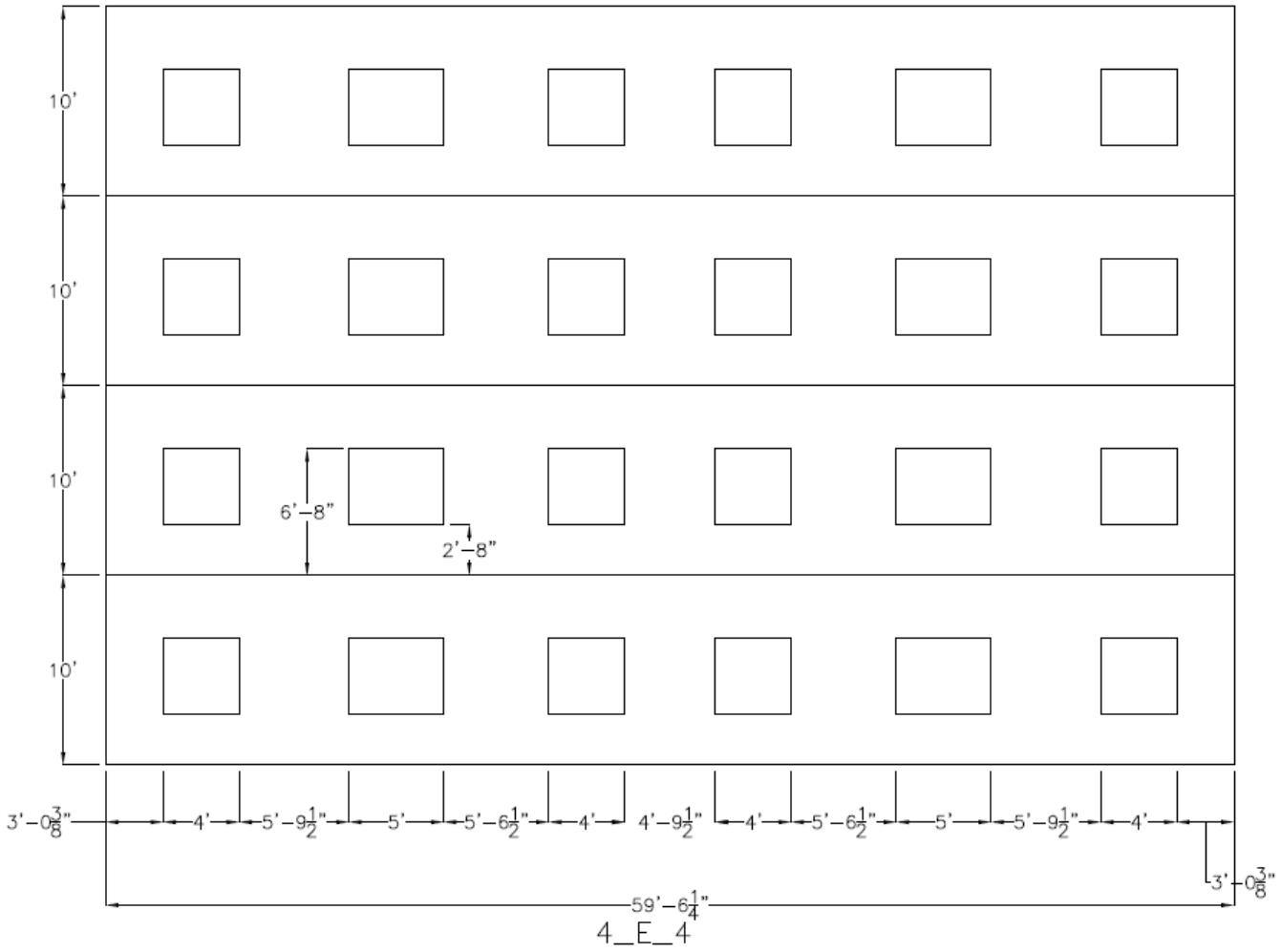


Figure B47. Index Building 4 extracted shear wall lines.

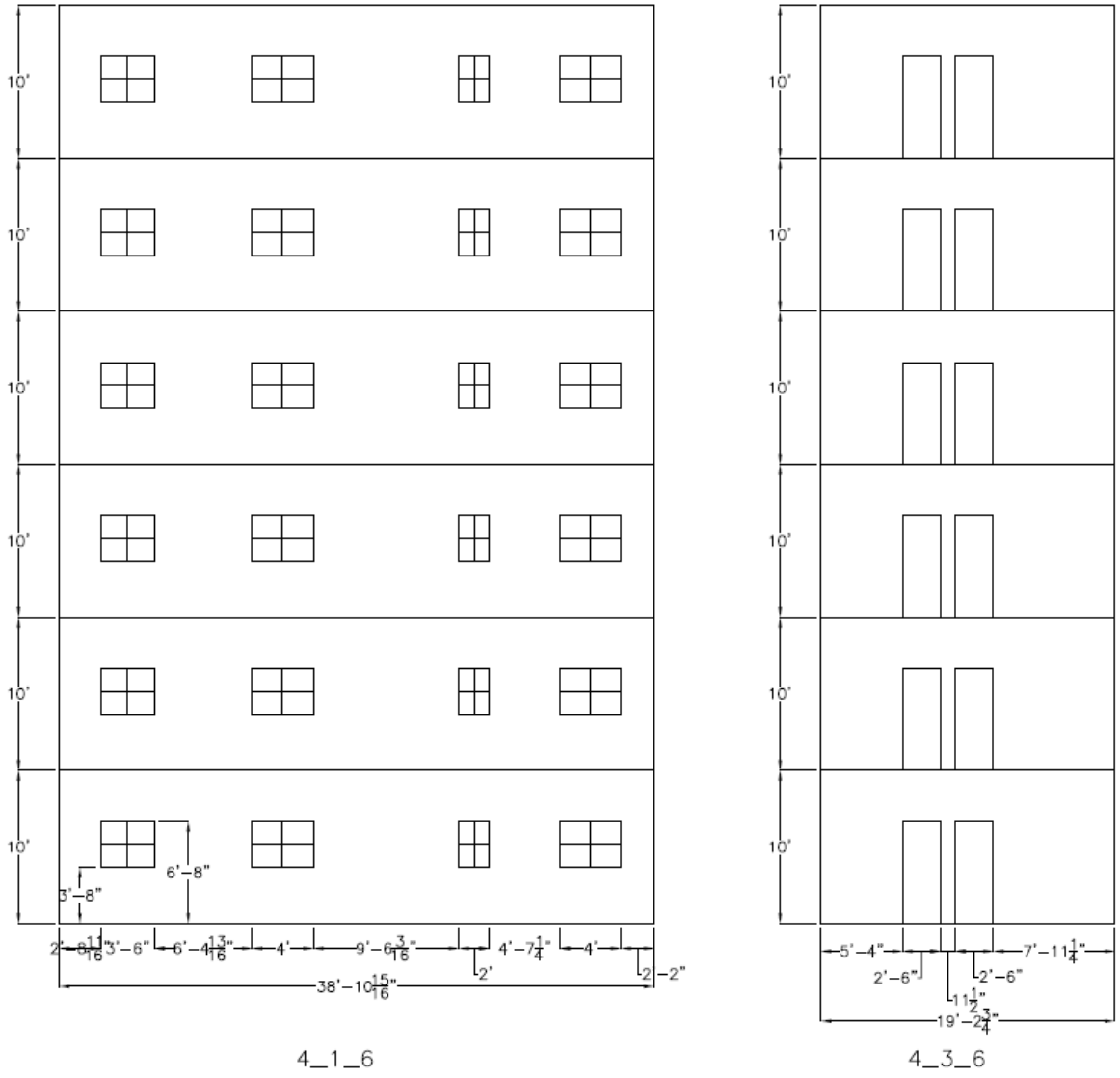


Figure B48. Index Building 4 extracted shear wall lines.

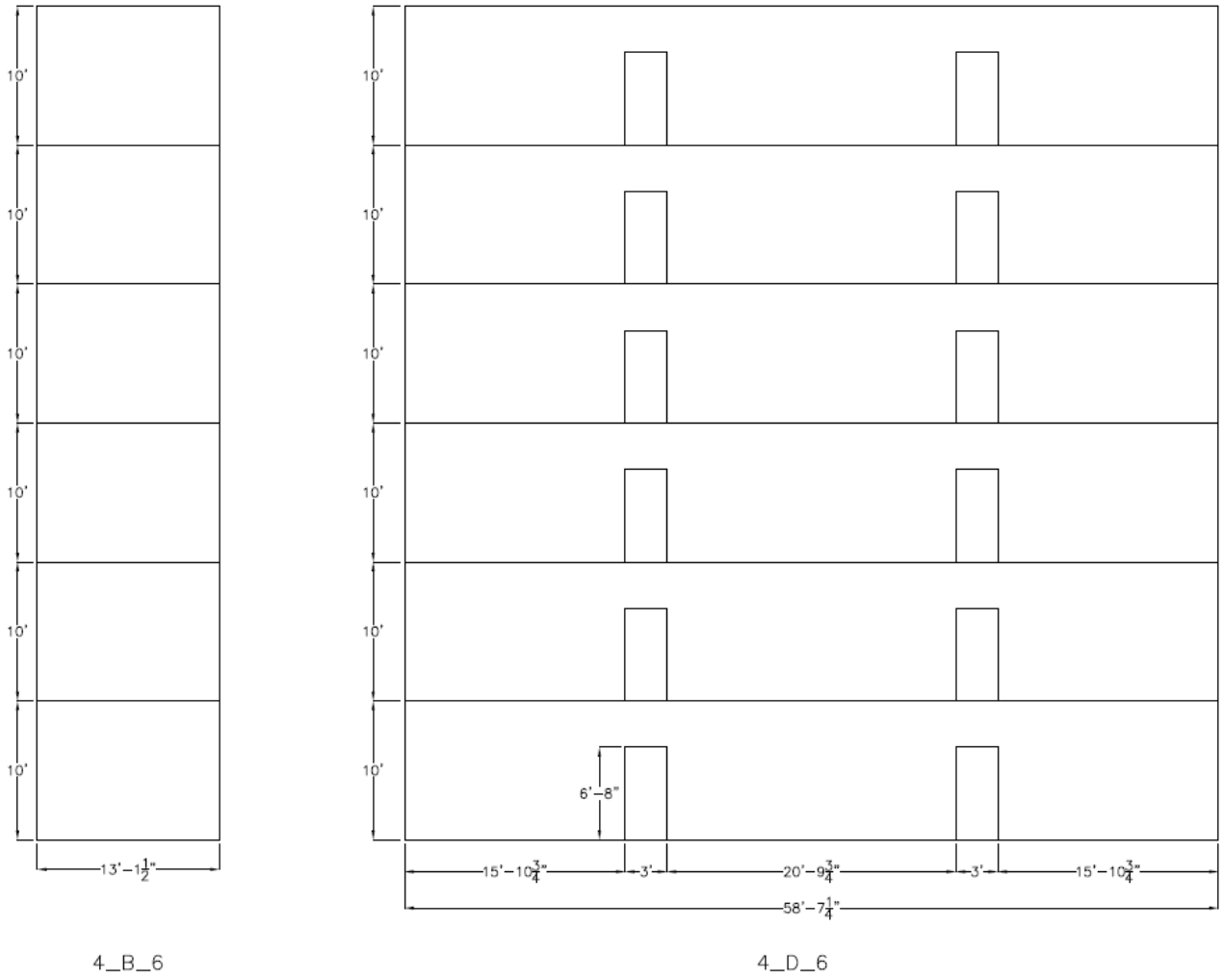


Figure B49. Index Building 4 extracted shear wall lines.

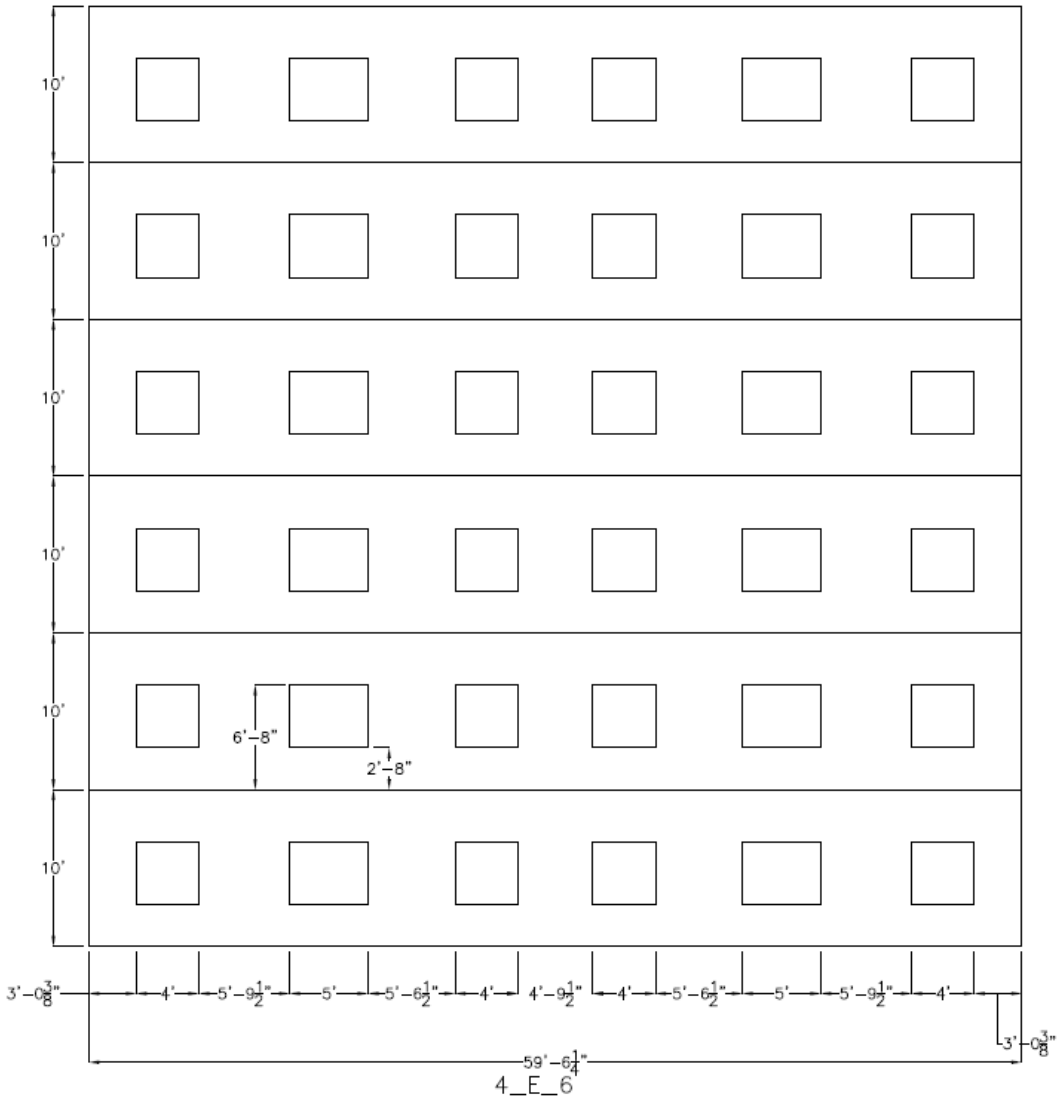


Figure B50. Index Building 4 extracted shear wall lines.

Table B39—Extracted archetypes, Index Building 4

Group No.		Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-2	2.5ft-20ft wall	Low aspect ratio panels	High	SDC D_{max}	4_3_ 6_1_LR_HG_ DX_LP	4
PG-6			Low		4_3_ 6_1_LR_LG_ DX_LP	16
PG-10		High aspect ratio panels	High		4_3_ 6_1_HR_HG_ DX_LP	28
PG-14			Low		4_3_ 6_1_HR_LG_ DX_LP	40
PG-18		Mixed aspect ratio	High		4_3_ 6_1_MR_HG_ DX_LP	52
					4_B_6_1_ MR_HG_ DX_LP	54
PG-22			Low		4_3_ 6_1_MR_LG_ DX_LP	64
					4_B_6_1_ MR_LG_ DX_LP	66
PG-17			High		4_3_ 4_1_MR_HG_ DX_SP	50
PG-21			Low		4_3_ 4_1_MR_LG_ DX_SP	62
PG-26	20ft-60ft wall	Low aspect ratio panels	High		4_E_6_2_ LR_HG_ DX_LP	76
PG-30			Low		4_E_6_2_ LR_LG_ DX_LP	88
PG-34		High aspect ratio panels	High		4_E_6_2_ HR_HG_ DX_LP	100
PG-38			Low		4_E_6_2_ HR_LG_ DX_LP	112
PG-42		Mixed aspect ratio	High		4_E_6_2_ MR_HG_ DX_LP	124
PG-46			Low		4_E_6_2_ MR_LG_ DX_LP	136

Table B40—Seismic weight detailed calculation, Index Building 4, 4-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)
Roof			2371					69391.27			**125107**	69391.27			**125107**
	4	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
3			2371					74054.23			**185486**	116998.97			**228431**
	3	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
2			2371					74054.23			**185486**	116998.97			**228431**
	2	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
1			2371					74054.23			**187820**	116998.97			**230765**
	1	10		196.83	1636.33	234.917	2349		55179	60922			55179	60922	
Ground			0												

Table B41—Seismic weight detailed calculation, Index Building 4, 6-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)
Roof			2371					69391.27			**125107**	69391.27			**125107**
	6	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
5			2371					69312.23			**185486**	116999			**228431**
	5	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
4			2371					69312.23			**185486**	116999			**228431**
	4	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
3			2371					69312.23			**185486**	116999			**228431**
	3	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
2			2371					69312.23			**185486**	1169989			**228431**
	2	10		196.83	1636.33	234.917	2169		55179	56254			55179	56254	
1			2371					69312.23			**187820**	116999			**230765**
	1	10		196.83	1636.33	234.917	2349		55179	60922			55179	60922	
Ground			0												

Table B42—Seismic weight summary, Index Building 4, 4-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		125.1	125.1
	10		
3		185.5	228.4
	10		
2		185.5	228.4
	10		
1		187.8	230.8
	10		
Ground			

Table B43—Seismic weight summary, Index Building 4, 6-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		125.1	125.1
	10		
5		185.5	228.4
	10		
4		185.5	228.4
	10		
3		185.5	228.4
	10		
2		185.5	228.4
	10		
1		187.8	230.8
	10		
Ground			

INPUT DATA

Total Height	h_n=	40.0	ft				
Total Weight	W=	684	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.32	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.4454					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	16,157					

DESIGN SUMMARY

Total base shear

V = 227.97

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
4		40.0	125	5,004	0.310	70.6		
	10.00						70.6	
3		30.0	186	5,565	0.344	78.5		706
	10.00						149.1	
2		20.0	186	3,710	0.230	52.3		2,197
	10.00						201.5	
1		10.0	188	1,878	0.116	26.5		4,212
	10.00						228.0	
		0.0						6,492

Seismic base shear calculation, low gravity, Index Building 4, 4-story, R = 3

INPUT DATA

Total Height	h_n=	40.0	ft				
Total Weight	W=	813	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.32	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.4454					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	18,732					

DESIGN SUMMARY

Total base shear

V = 270.90

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
4		40.0	125	5,004	0.267	72.4		
	10.00						72.4	
3		30.0	228	6,852	0.366	99.1		724
	10.00						171.5	
2		20.0	228	4,568	0.244	66.1		2,438
	10.00						237.5	
1		10.0	231	2,308	0.123	33.4		4,813
	10.00						270.9	
		0.0						7,522

Seismic base shear calculation, high gravity, Index Building 4, 4-story, R = 3

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	1,055	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^X$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	42,708					

DESIGN SUMMARY

Total base shear V = 349.52

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	125	9,280	0.217	75.9		
	10.00						75.9	
5		50.0	186	11,359	0.266	93.0		759
	10.00						168.9	
4		40.0	186	8,983	0.210	73.5		2,449
	10.00						242.4	
3		30.0	186	6,637	0.155	54.3		4,873
	10.00						296.7	
2		20.0	186	4,333	0.101	35.5		7,840
	10.00						332.2	
1		10.0	188	2,116	0.050	17.3		11,162
	10.00						349.5	
		0.0						14,657

Seismic base shear calculation, low gravity, Index Building 4, 6-story, R = 3

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	1,270	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^X$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	50,434					

DESIGN SUMMARY

Total base shear V = 420.62

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	125	9,280	0.184	77.4		
	10.00						77.4	
5		50.0	228	13,986	0.277	116.6		774
	10.00						194.0	
4		40.0	228	11,060	0.219	92.2		2,714
	10.00						286.3	
3		30.0	228	8,173	0.162	68.2		5,577
	10.00						354.4	
2		20.0	228	5,335	0.106	44.5		9,122
	10.00						398.9	
1		10.0	231	2,600	0.052	21.7		13,111
	10.00						420.6	
		0.0						17,317

Seismic base shear calculation, high gravity, Index Building 4, 6-story, R = 3

Table B44—Tributary load calculation, Index Building 4, 4-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
	1	205.0	0.086		6.11		6.26
	2	392.0	0.165		11.68		11.97
	3	**392.0**	**0.165**		**11.68**		**11.97**
Story 4	4	392.0	0.165	70.60	11.68	72.37	11.97
	5	392.0	0.165		11.68		11.97
	6	392.0	0.165		11.68		11.97
	7	205.0	0.086		6.11		6.26
	1	205.0	0.086		12.90		14.83
	2	392.0	0.165		24.67		28.36
	3	**392.0**	**0.165**		**24.67**		**28.36**
Story 3	4	392.0	0.165	149.12	24.67	171.46	28.36
	5	392.0	0.165		24.67		28.36
	6	392.0	0.165		24.67		28.36
	7	205.0	0.086		12.90		14.83
	1	205.0	0.086		17.43		20.55
	2	392.0	0.165		33.32		39.29
	3	**392.0**	**0.165**		**33.32**		**39.29**
Story 2	4	392.0	0.165	201.47	33.32	237.52	39.29
	5	392.0	0.165		33.32		39.29
	6	392.0	0.165		33.32		39.29
	7	205.0	0.086		17.43		20.55
	1	205.0	0.086		19.72		23.43
	2	392.0	0.165		37.71		44.81
	3	**392.0**	**0.165**		**37.71**		**44.81**
Story 1	4	392.0	0.165	227.97	37.71	270.90	44.81
	5	392.0	0.165		37.71		44.81
	6	392.0	0.165		37.71		44.81
	7	205.0	0.086		19.72		23.43

Table B45—Tributary load calculation, Index Building 4, 6-story, N-S direction, R = 3*

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft²)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	1	205.0	0.086	75.95	6.57	77.39	6.69
	2	392.0	0.165		12.56		12.80
	3	**392.0**	**0.165**		**12.56**		**12.80**
	4	392.0	0.165		12.56		12.80
	5	392.0	0.165		12.56		12.80
	6	392.0	0.165		12.56		12.80
	7	205.0	0.086		6.57		6.69
Story 5	1	205.0	0.086	168.91	14.61	194.04	16.78
	2	392.0	0.165		27.94		32.09
	3	**392.0**	**0.165**		**27.94**		**32.09**
	4	392.0	0.165		27.94		32.09
	5	392.0	0.165		27.94		32.09
	6	392.0	0.165		27.94		32.09
	7	205.0	0.086		14.61		16.78
Story 4	1	205.0	0.086	242.42	20.97	286.28	24.76
	2	392.0	0.165		40.10		47.35
	3	**392.0**	**0.165**		**40.10**		**47.35**
	4	392.0	0.165		40.10		47.35
	5	392.0	0.165		40.10		47.35
	6	392.0	0.165		40.10		47.35
	7	205.0	0.086		20.97		24.76
Story 3	1	205.0	0.086	296.74	25.67	354.44	30.66
	2	392.0	0.165		49.08		58.62
	3	**392.0**	**0.165**		**49.08**		**58.62**
	4	392.0	0.165		49.08		58.62
	5	392.0	0.165		49.08		58.62
	6	392.0	0.165		49.08		58.62
	7	205.0	0.086		25.67		30.66
Story 2	1	205.0	0.086	332.20	28.73	398.93	34.51
	2	392.0	0.165		54.95		65.98
	3	**392.0**	**0.165**		**54.95**		**65.98**
	4	392.0	0.165		54.95		65.98
	5	392.0	0.165		54.95		65.98
	6	392.0	0.165		54.95		65.98
	7	205.0	0.086		28.73		34.51
Story 1	1	205.0	0.086	349.52	30.23	420.62	36.38
	2	392.0	0.165		57.81		69.57
	3	**392.0**	**0.165**		**57.81**		**69.57**
	4	392.0	0.165		57.81		69.57
	5	392.0	0.165		57.81		69.57
	6	392.0	0.165		57.81		69.57
	7	205.0	0.086		30.23		36.38

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B46—Tributary load calculation, Index Building 4, 6-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft²)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	312.0	0.135	75.95	10.28	77.39	10.48
	B	**340.0**	**0.148**		**11.21**		**11.42**
	C	271.0	0.118		8.93		9.10
	D	781.0	0.339		25.74		26.23
	E	**600.0**	**0.260**		**19.78**		**20.15**
Story 5	A	312.0	0.135	168.91	22.87	194.04	26.28
	B	**340.0**	**0.148**		**24.93**		**28.63**
	C	271.0	0.118		19.87		22.82
	D	781.0	0.339		57.26		65.77
	E	**600.0**	**0.260**		**43.99**		**50.53**
Story 4	A	312.0	0.135	242.42	32.83	286.28	38.77
	B	**340.0**	**0.148**		**35.77**		**42.25**
	C	271.0	0.118		28.51		33.67
	D	781.0	0.339		82.17		97.04
	E	**600.0**	**0.260**		**63.13**		**74.55**
Story 3	A	312.0	0.135	296.74	40.18	354.44	48.00
	B	**340.0**	**0.148**		**43.79**		**52.30**
	C	271.0	0.118		34.90		41.69
	D	781.0	0.339		100.59		120.15
	E	**600.0**	**0.260**		**77.28**		**92.30**
Story 2	A	312.0	0.135	332.20	44.99	398.93	54.02
	B	**340.0**	**0.148**		**49.02**		**58.87**
	C	271.0	0.118		39.07		46.92
	D	781.0	0.339		112.61		135.23
	E	**600.0**	**0.260**		**86.51**		**103.89**
Story 1	A	312.0	0.135	349.52	47.33	420.62	56.96
	B	**340.0**	**0.148**		**51.58**		**62.07**
	C	271.0	0.118		41.11		49.47
	D	781.0	0.339		118.48		142.58
	E	**600.0**	**0.260**		**91.02**		**109.54**

* Seismic load for shear wall line E is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B47—Archetype 4 (4_3_ 6_1_LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	4	3/8	0.25	1.43	640.1	-	-	-
5	1	S	5	4.125	3	A3	4	D	2084.0	4	3/4	0.27	3.29	1604.7	-	-	-
4	1	S	5	4.125	3	A3	5	D	2605.0	4	1	0.36	3.70	2367.6	-	-	-
3	1	S	5	6.875	5	A3	6	D	3126.0	4	1 1/4	0.33	4.89	2931.3	-	-	-
2	1	S	5	6.875	5	A3	7	D	3647.0	4	1 1/2	0.35	5.28	3299.2	-	-	-
1	1	S	5	6.875	5	A3	7	D	3647.0	4	1 3/4	0.36	5.35	3478.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.13	3	A3	3	S	781.5	4	3/8	0.25	1.43	640.1	7.82	6.40	1.22
5	1	S	5	4.125	3	A3	4	D	2084.0	4	3/4	0.27	3.29	1604.7	20.84	16.05	1.30
4	1	S	5	4.125	3	A3	5	D	2605.0	4	1	0.36	3.70	2367.6	26.05	23.68	1.10
3	1	S	5	6.875	5	A3	6	D	3126.0	4	1 1/4	0.33	4.89	2931.2	31.26	29.31	1.07
2	1	S	5	6.875	5	A3	7	D	3647.0	4	1 1/2	0.35	5.28	3299.2	36.47	32.99	1.11
1	1	S	5	6.875	5	A3	7	D	3647.0	4	1 3/4	0.36	5.35	3478.5	36.47	34.79	1.05

*Values shown are for reinforced option and before applying C_d.

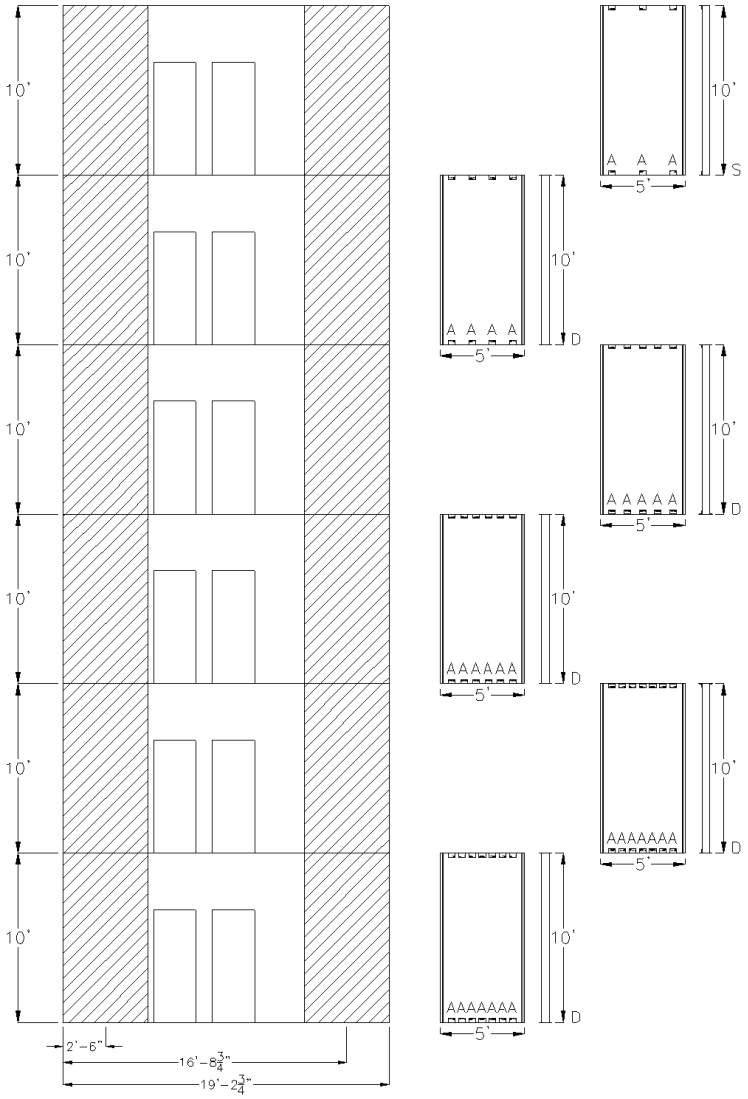


Figure B51. Archetype 4.

Table B48—Archetype 16 (4_3_ 6_1_LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	4	3/8	0.24	1.43	628.1	-	-	-
5	1	S	5	4.125	3	A3	3	D	1563.0	4	3/4	0.25	3.05	1396.9	-	-	-
4	1	S	5	4.125	3	A3	4	D	2084.0	4	1	0.31	3.55	2004.8	-	-	-
3	1	S	5	4.125	3	A3	5	D	2605.0	4	1 1/4	0.36	3.82	2454.1	-	-	-
2	1	S	5	4.125	3	A3	6	D	3126.0	4	1 1/2	0.41	3.75	2747.3	-	-	-
1	1	S	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.32	5.00	2890.5	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	4	3/8	0.24	1.43	628.1	7.82	6.28	1.24
5	1	S	5	4.125	3	A3	3	D	1563.0	4	3/4	0.25	3.05	1396.9	15.63	13.97	1.12
4	1	S	5	4.125	3	A3	4	D	2084.0	4	1	0.31	3.55	2004.8	20.84	20.05	1.04
3	1	S	5	4.125	3	A3	5	D	2605.0	4	1 1/4	0.36	3.82	2454.1	26.05	24.54	1.06
2	1	S	5	4.125	3	A3	6	D	3126.0	4	1 1/2	0.41	3.75	2747.3	31.26	27.47	1.14
1	1	S	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.32	5.00	2890.6	31.26	28.91	1.08

*Values shown are for reinforced option and before applying C_d.

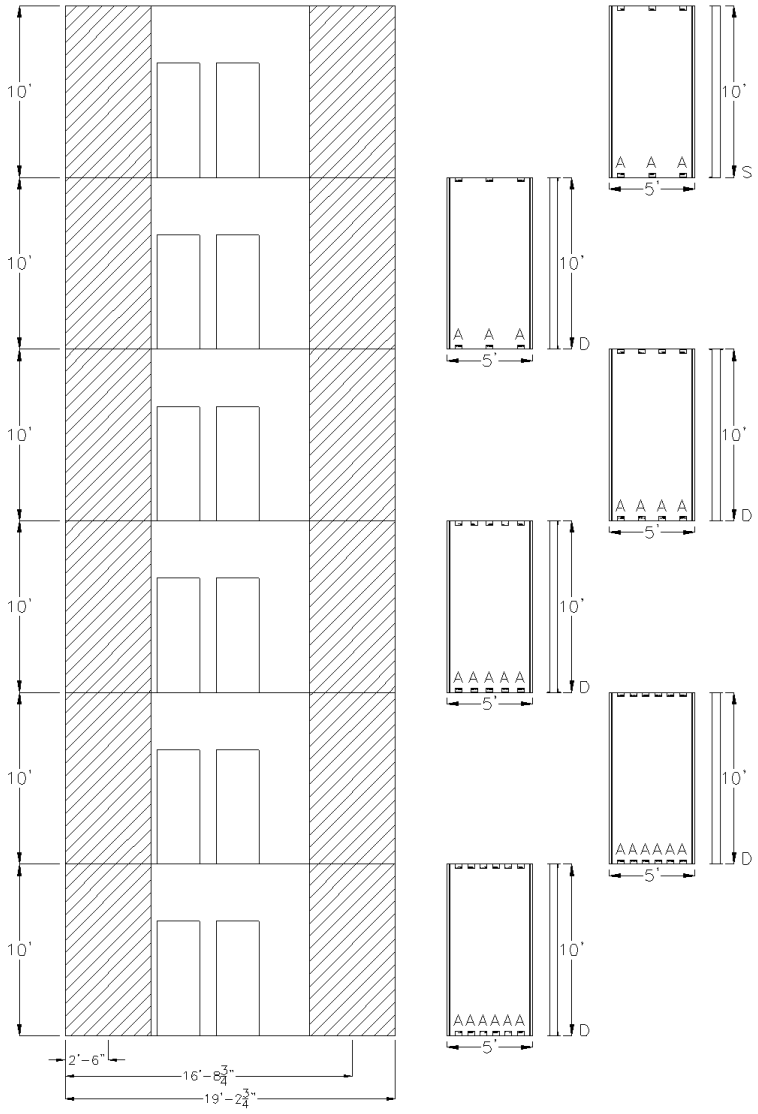


Figure B52. Archetype 16.

Table B49—Archetype 28 (4_3_ 6_1_HR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	6	3/8	0.28	1.56	522.2	-	-	-
5	3	M	2.5	4.125	3	A3	2	D	2084.0	6	5/8	0.53	2.39	1515.1	-	-	-
4	3	M	2.5	4.125	3	A3	2	D	2084.0	6	1 1/8	0.63	2.62	1985.0	-	-	-
3	3	M	2.5	6.875	5	A3	3	D	3126.0	6	1 1/4	0.58	3.81	2667.8	-	-	-
2	3	M	2.5	6.875	5	A3	3	D	3126.0	6	1 1/2	0.65	3.85	3017.7	-	-	-
1	3	M	2.5	6.875	5	A3	3	D	3126.0	6	1 1/2	0.66	3.70	2921.7	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.28	0.99	496.8	13.03	6.40	2.03
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.53	0.99	936.8	20.84	16.05	1.30
4	2	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.63	1.54	1757.7	26.05	23.68	1.10
3	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.58	1.77	1860.8	33.87	29.31	1.16
2	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.65	1.76	2071.9	33.87	32.99	1.03
1	2	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/2	0.66	2.17	2574.6	39.08	34.79	1.12

*Values shown are for reinforced option and before applying C_d.

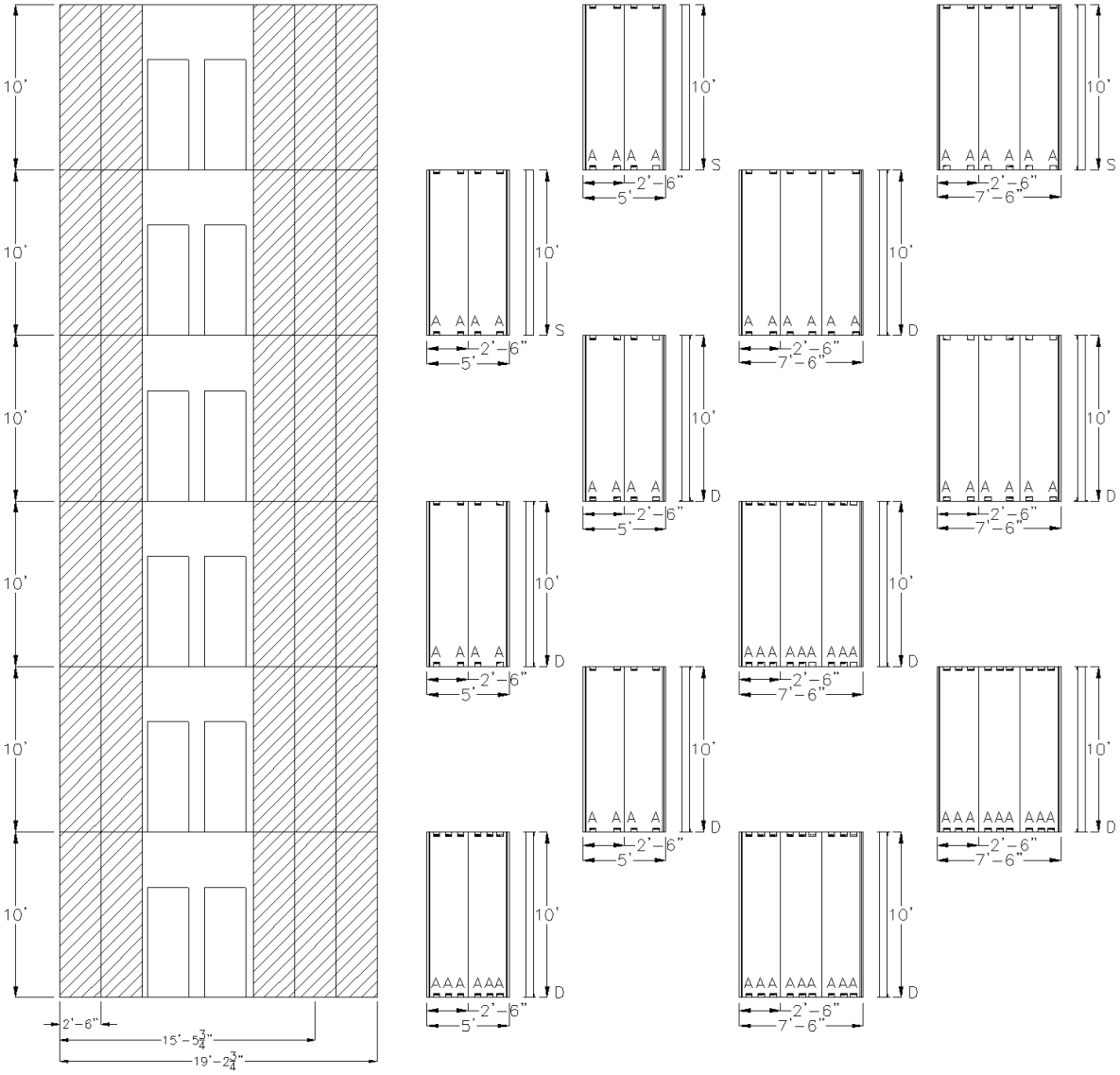


Figure B53. Archetype 28.

Table B50—Archetype 40 (4_3_ 6_1_HR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.27	0.99	476.1	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.50	0.99	888.8	-	-	-
4	2	M	2.5	6.875	5	A3	2	S	1042.0	4	3/4	0.53	1.09	1030.1	-	-	-
3	2	M	2.5	6.875	5	A3	2	S	1042.0	4	7/8	0.54	1.08	1041.6	-	-	-
2	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.54	1.72	1688.8	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.58	1.71	1802.0	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.27	1.62	520.1	13.03	6.28	2.07
5	3	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.50	2.12	1270.0	16.93	13.97	1.21
4	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.53	3.15	1986.4	20.84	20.05	1.04
3	3	M	2.5	6.875	5	A3	3	D	3126.0	4	2	0.54	4.00	2577.7	28.66	24.54	1.17
2	3	M	2.5	6.875	5	A3	3	D	3126.0	4	2	0.54	3.88	2537.3	33.87	27.47	1.23
1	3	M	2.5	6.875	5	A3	3	D	3126.0	4	2	0.58	3.78	2652.7	33.87	28.91	1.17

*Values shown are for reinforced option and before applying C_d.

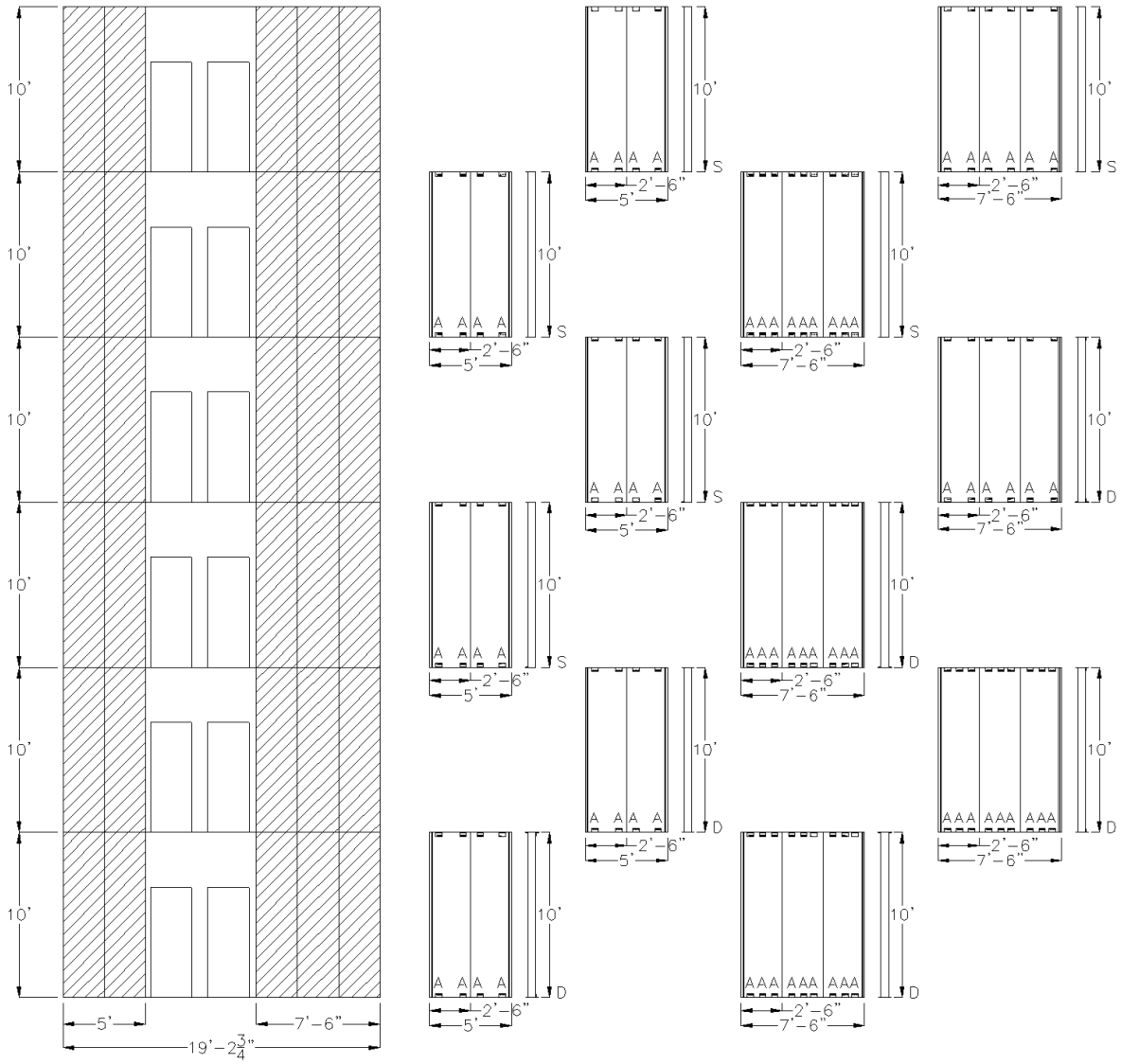


Figure B54. Archetype 40.

Table B51—Archetype 52 (4_3_ 6_1_MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	2	S	521.0	1	1/2	0.30	0.80	436.7	-	-	-
5	1	S	5	4.125	3	A3	3	D	1563.0	4	1/2	0.43	1.99	1523.3	-	-	-
4	1	S	5	6.875	5	A3	5	D	2605.0	4	3/4	0.42	3.29	2505.5	-	-	-
3	1	S	5	9.625	7	A3	5	D	2605.0	4	7/8	0.45	3.16	2535.9	-	-	-
2	1	S	5	12.375	9	A3	5	D	2605.0	4	1	0.45	3.15	2553.0	-	-	-
1	1	S	5	12.375	9	A3	6	D	3126.0	4	1 1/8	0.47	3.42	2882.3	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.30	1.55	562.3	10.42	6.40	1.63
5	3	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.43	2.20	1124.1	19.54	16.05	1.22
4	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1	0.42	2.93	1486.4	28.66	23.68	1.21
3	3	M	2.5	9.625	7	A3	3	D	3126.0	4	1 1/4	0.45	4.14	2217.7	36.47	29.31	1.24
2	3	M	2.5	12.375	9	A3	3	D	3126.0	4	2	0.45	4.99	2696.9	36.47	32.99	1.11
1	3	M	2.5	12.375	9	A3	3	D	3126.0	4	2	0.47	4.83	2716.5	39.08	34.79	1.12

*Values shown are for reinforced option and before applying C_d.

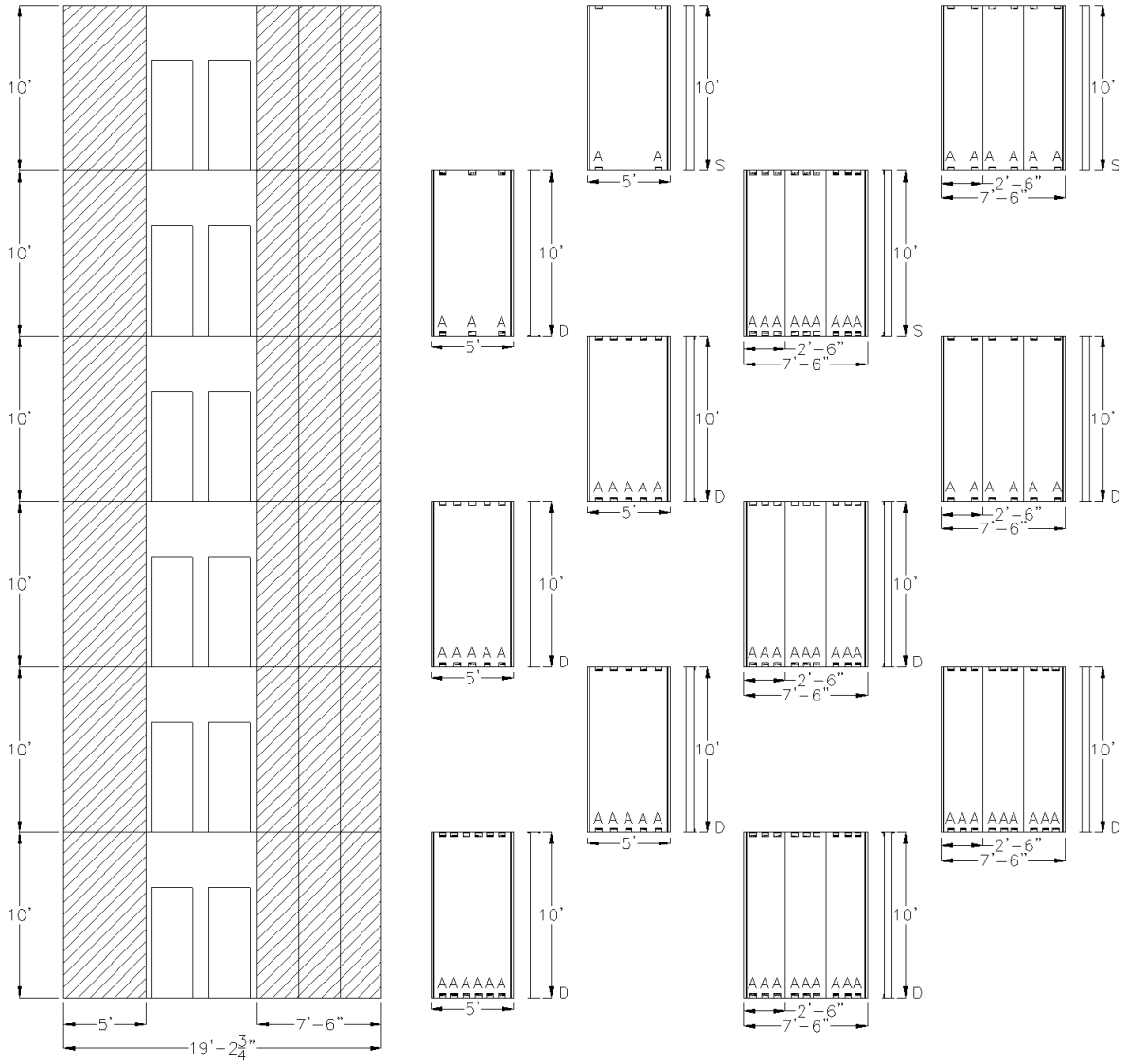


Figure B55. Archetype 52.

Table B52—Archetype 64 (4_3_ 6_1_MR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	2	S	521.0	2	1/2	0.23	1.23	520.8	-	-	-
5	1	S	5	4.125	3	A3	6	S	1563.0	4	1/2	0.43	1.88	1457.2	-	-	-
4	1	S	5	4.125	3	A3	5	D	2605.0	4	3/4	0.47	2.68	2258.3	-	-	-
3	1	S	5	6.875	5	A3	5	D	2605.0	4	7/8	0.47	2.87	2423.1	-	-	-
2	1	S	5	6.875	5	A3	6	D	3126.0	4	1	0.52	2.99	2782.4	-	-	-
1	1	S	5	6.875	5	A3	6	D	3126.0	4	1 1/8	0.55	2.91	2901.1	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.23	1.74	490.2	10.42	6.28	1.66
5	3	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.43	1.72	891.0	15.63	13.97	1.12
4	3	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.47	2.07	1167.6	24.75	20.05	1.23
3	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.47	2.94	1656.7	28.66	24.54	1.17
2	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.52	2.92	1808.1	31.26	27.47	1.14
1	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.55	2.89	1920.0	31.26	28.91	1.08

*Values shown are for reinforced option and before applying C_d.

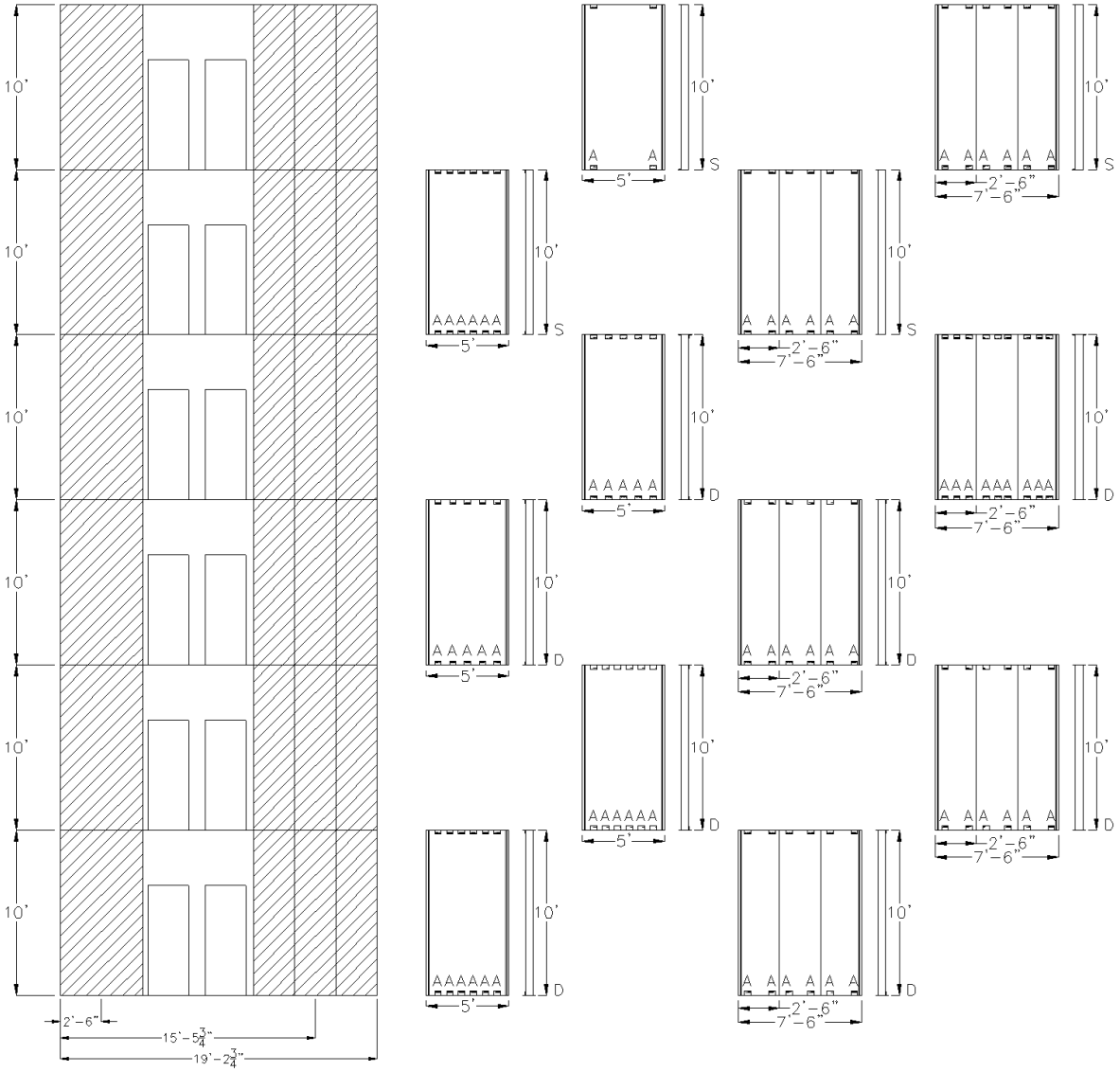


Figure B56. Archetype 64.

Table B53—Archetype 54 (4_B_6_1_ MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.22	1.63	423.0	-	-	-
5	3	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.45	1.64	883.8	-	-	-
4	3	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.54	2.49	1624.2	-	-	-
3	3	M	2.5	4.125	3	A3	2	D	2084.0	4	1 1/2	0.57	2.60	1766.5	-	-	-
2	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/2	0.53	3.07	1965.4	-	-	-
1	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/2	0.51	2.92	1793.8	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	2	S	521.0	4	3/8	0.22	1.30	507.7	10.42	5.71	1.82
5	1	S	5	4.125	3	A3	3	D	1563.0	4	1/2	0.45	1.90	1537.6	15.63	14.32	1.09
4	1	S	5	4.125	3	A3	4	D	2084.0	4	5/8	0.54	1.83	1788.3	26.05	21.12	1.23
3	1	S	5	4.125	3	A3	5	D	2605.0	4	7/8	0.57	2.53	2580.7	28.66	26.15	1.10
2	1	S	5	6.875	5	A3	6	D	3126.0	4	1	0.53	3.06	2938.9	31.26	29.44	1.06
1	1	S	5	6.875	5	A3	7	D	3647.0	4	1 1/4	0.51	3.81	3516.4	33.87	31.04	1.09

*Values shown are for reinforced option and before applying C_d.

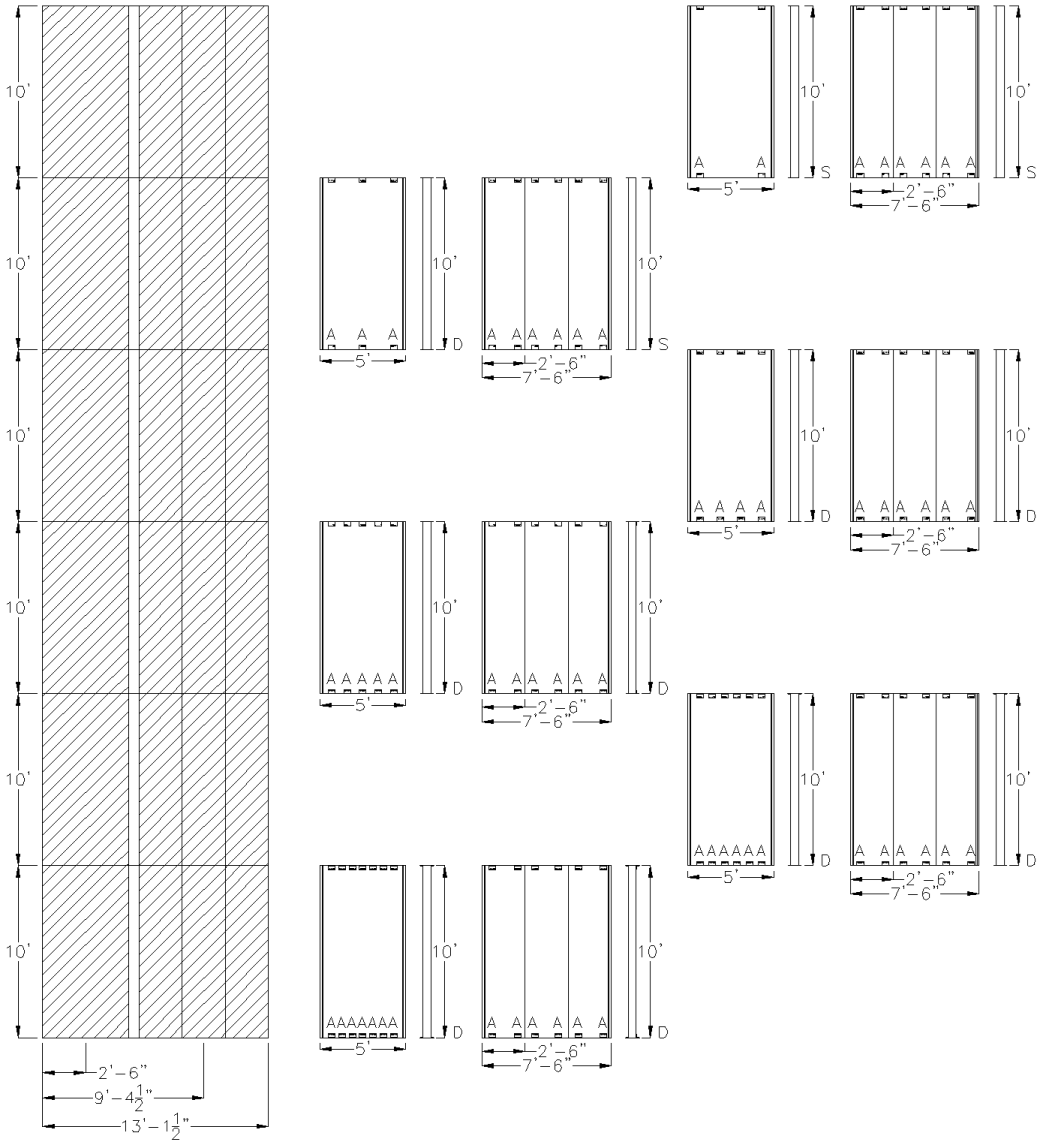


Figure B57. Archetype 54.

Table B54—Archetype 66 (4_B_6_1_ MR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.21	1.63	404.7	-	-	-
5	3	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.44	1.64	862.3	-	-	-
4	3	M	2.5	6.875	5	A3	2	S	1042.0	4	3/4	0.47	1.83	1038.1	-	-	-
3	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1	0.49	2.86	1671.9	-	-	-
2	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.51	2.94	1794.2	-	-	-
1	3	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.53	2.89	1847.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	2	1/2	0.21	1.38	513.6	11.72	5.60	2.09
5	1	S	5	4.125	3	A3	5	S	1302.5	2	5/8	0.44	1.52	1199.1	14.33	12.46	1.15
4	1	S	5	6.875	5	A3	4	D	2084.0	2	7/8	0.47	2.37	2020.3	18.24	17.89	1.02
3	1	S	5	6.875	5	A3	4	D	2084.0	4	3/4	0.49	2.13	1871.1	26.05	21.89	1.19
2	1	S	5	6.875	5	A3	5	D	2605.0	4	7/8	0.51	2.41	2210.9	28.66	24.51	1.17
1	1	S	5	6.875	5	A3	5	D	2605.0	4	1	0.53	2.49	2387.0	28.66	25.79	1.11

*Values shown are for reinforced option and before applying C_d.

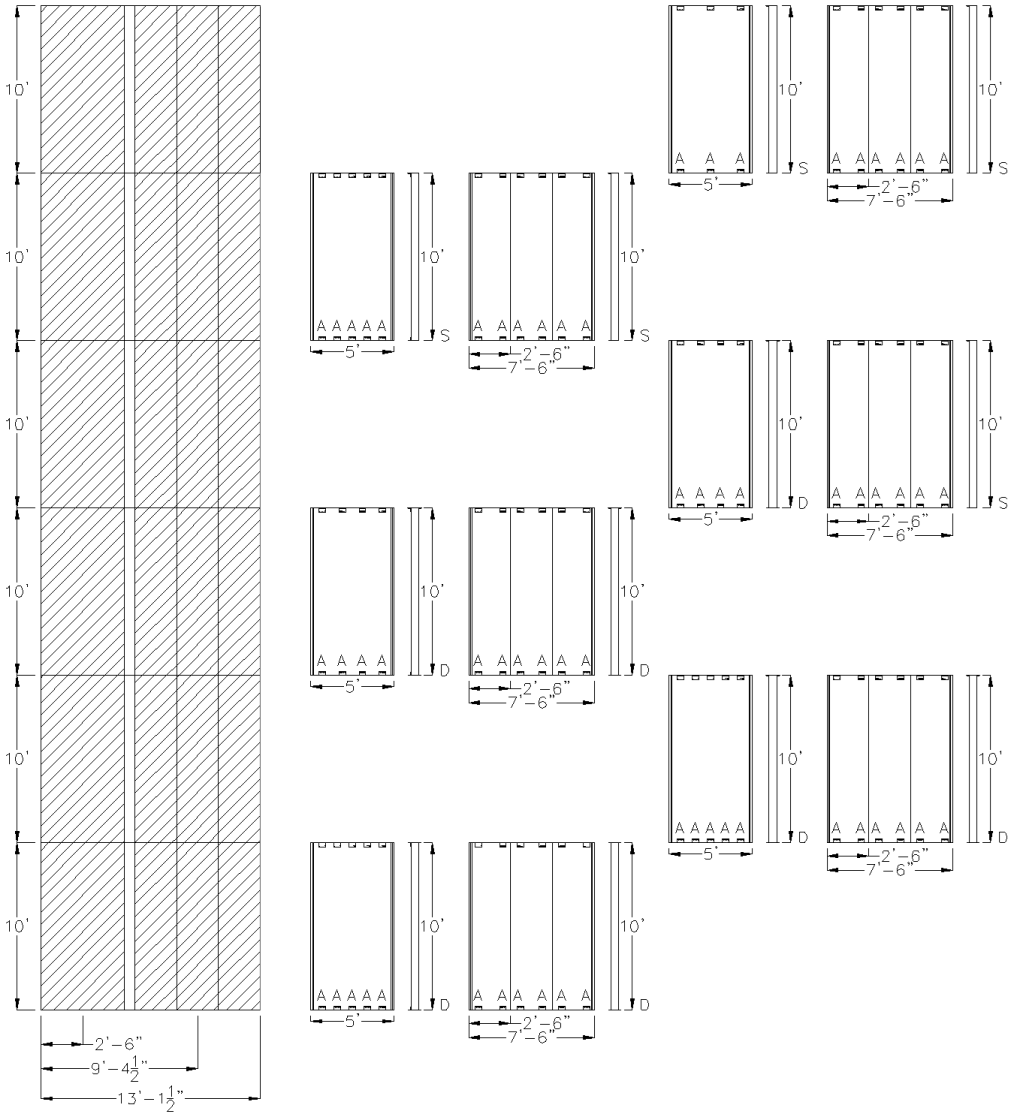


Figure B58. Archetype 66.

Table B55—Archetype 50 (4_3_ 4_1_MR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.68	1.01	826.8	-	-	-
3	3	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.94	1.76	1976.3	-	-	-
2	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/4	0.91	2.57	2818.9	-	-	-
1	3	M	2.5	12.375	7	A3	3	D	3126.0	4	1 1/2	0.82	3.13	3086.9	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	1	S	5	4.125	3	A3	5	S	1302.5	4	1/2	0.68	0.94	1153.7	14.33	11.97	1.20
3	1	S	5	4.125	3	A3	6	D	3126.0	4	3/4	0.94	1.61	2707.5	31.26	28.36	1.10
2	1	S	5	6.875	5	A3	8	D	4168.0	4	1 1/8	0.91	2.20	3628.8	44.29	39.29	1.13
1	1	S	5	12.375	7	A3	9	D	4689.0	4	1 1/2	0.82	2.93	4331.1	46.89	44.81	1.05

*Values shown are for unreinforced option and before applying C_d.

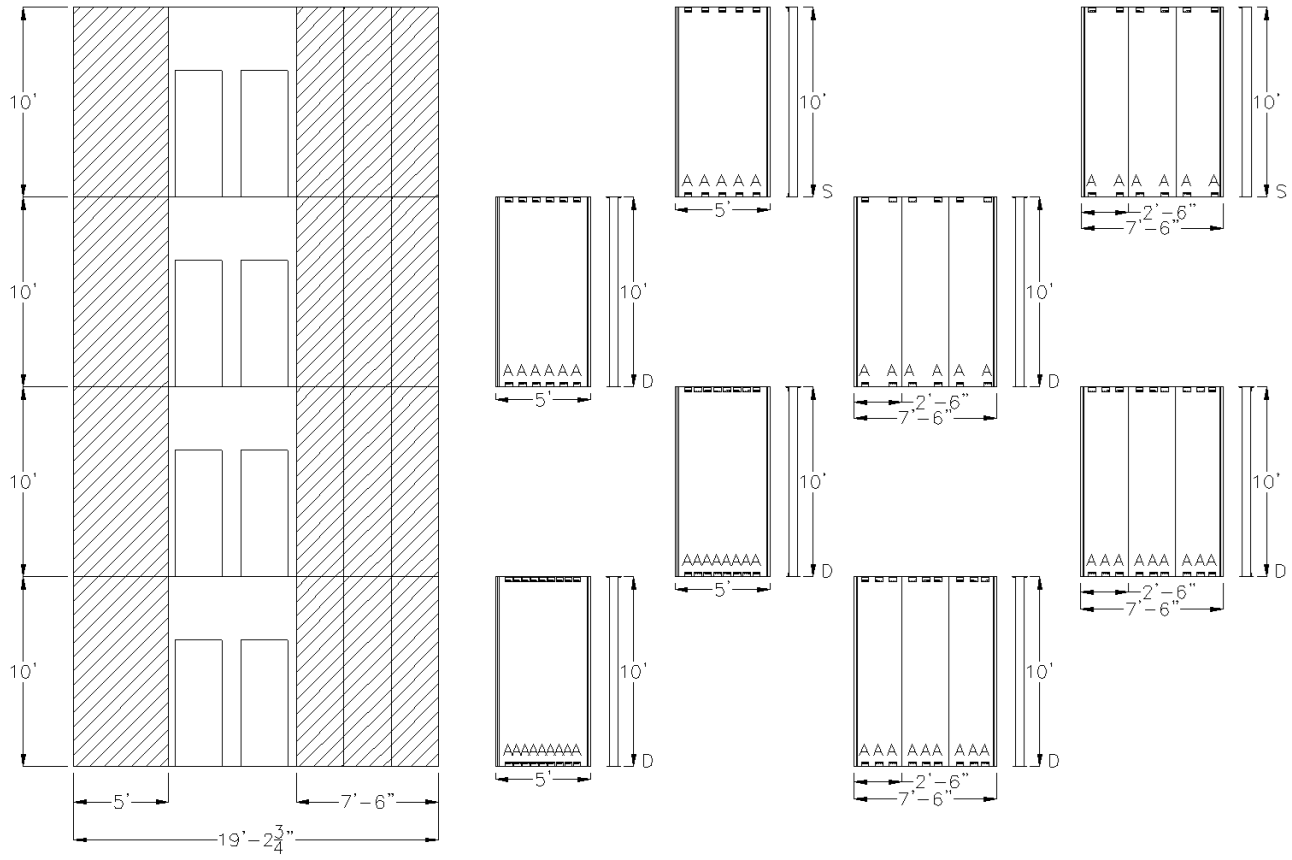


Figure B59. Archetype 50.

Table B56—Archetype 62 (4_3_ 4_1_MR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.37	1.62	708.1	-	-	-
3	3	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.49	2.08	1222.2	-	-	-
2	3	M	2.5	4.125	3	A3	3	D	3126.0	4	1 1/8	0.63	2.91	2186.2	-	-	-
1	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/8	0.60	3.69	2643.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	1	S	5	4.125	3	A3	5	S	1302.5	2	5/8	0.37	1.94	1273.5	14.33	11.68	1.23
3	1	S	5	4.125	3	A3	6	D	3126.0	2	1 1/8	0.49	3.51	3099.8	27.35	24.67	1.11
2	1	S	5	4.125	3	A3	7	D	3647.0	2	1 3/8	0.63	3.01	3385.3	41.68	33.32	1.25
1	1	S	5	6.875	5	A3	7	D	3647.0	2	1 1/2	0.60	3.32	3575.7	41.68	37.71	1.11

*Values shown are for reinforced option and before applying C_d.

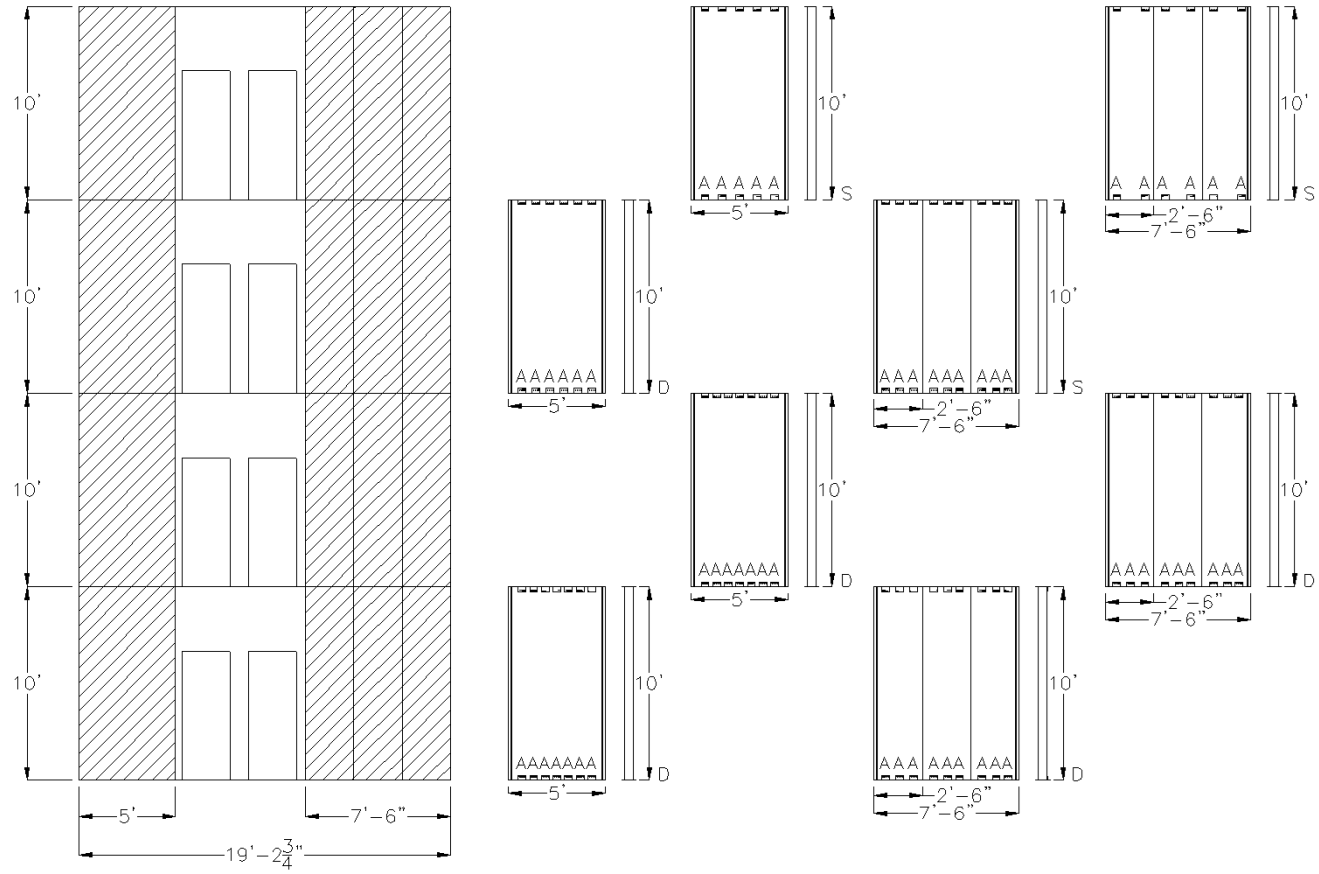


Figure B60. Archetype 62.

Table B57—Archetype 76 (4_E_6_2_ LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	5	S	5	4.125	3	A3	3	S	781.5	4	1/2	0.08	2.82	403.1	19.54	10.08	1.94
5	5	S	5	4.125	3	A3	4	S	1042.0	4	5/8	0.19	2.92	1010.6	26.05	25.27	1.03
4	5	S	5	4.125	3	A3	3	D	1563.0	4	3/4	0.28	2.97	1491.0	39.08	37.28	1.05
3	5	S	5	4.125	3	A3	4	D	2084.0	4	1	0.31	3.31	1846.0	52.10	46.15	1.13
2	5	S	5	4.125	3	A3	4	D	2084.0	4	1 1/4	0.36	3.20	2077.8	52.10	51.94	1.00
1	5	S	5	6.875	5	A3	5	D	2605.0	4	1 1/2	0.27	4.57	2190.7	65.13	54.77	1.19

*Values shown are for reinforced option and before applying C_d.

Table B58—Archetype 88 (4_E_6_2_ LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	5	S	5	4.125	3	A3	2	S	521.0	2	1/2	0.12	1.83	395.5	13.03	9.89	1.32
5	5	S	5	4.125	3	A3	4	S	1042.0	2	3/4	0.20	2.49	879.7	26.05	21.99	1.18
4	5	S	5	4.125	3	A3	5	S	1302.5	2	1	0.26	2.68	1262.6	32.56	31.57	1.03
3	5	S	5	4.125	3	A3	3	D	1563.0	2	1 1/4	0.31	2.81	1545.5	39.08	38.64	1.01
2	5	S	5	6.875	5	A3	4	D	2084.0	2	1 1/2	0.26	3.64	1730.2	52.10	43.26	1.20
1	5	S	5	6.875	5	A3	4	D	2084.0	2	1 3/4	0.27	3.72	1820.4	52.10	45.51	1.14

*Values shown are for reinforced option and before applying C_d.

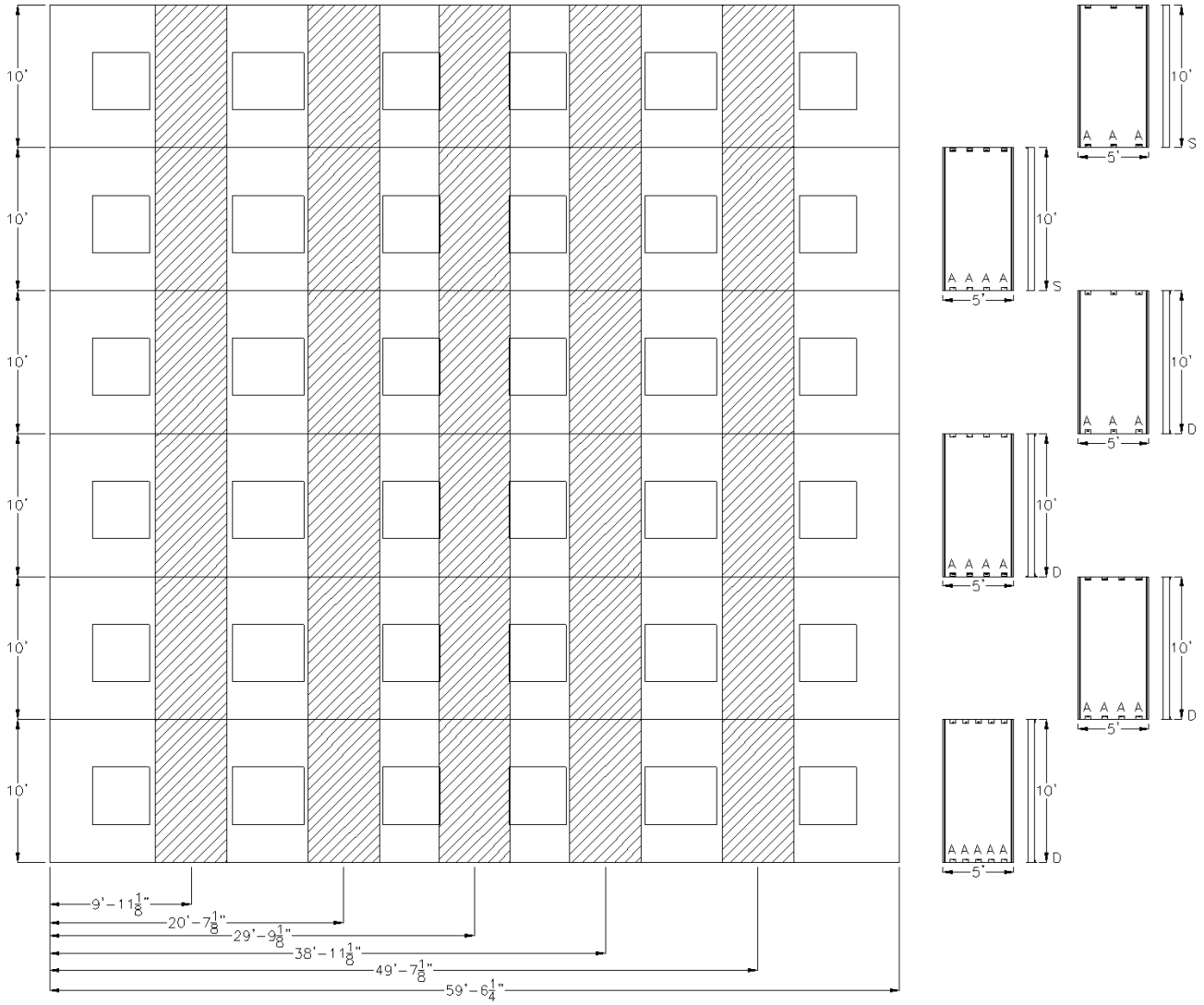


Figure B61. Archetype 76.

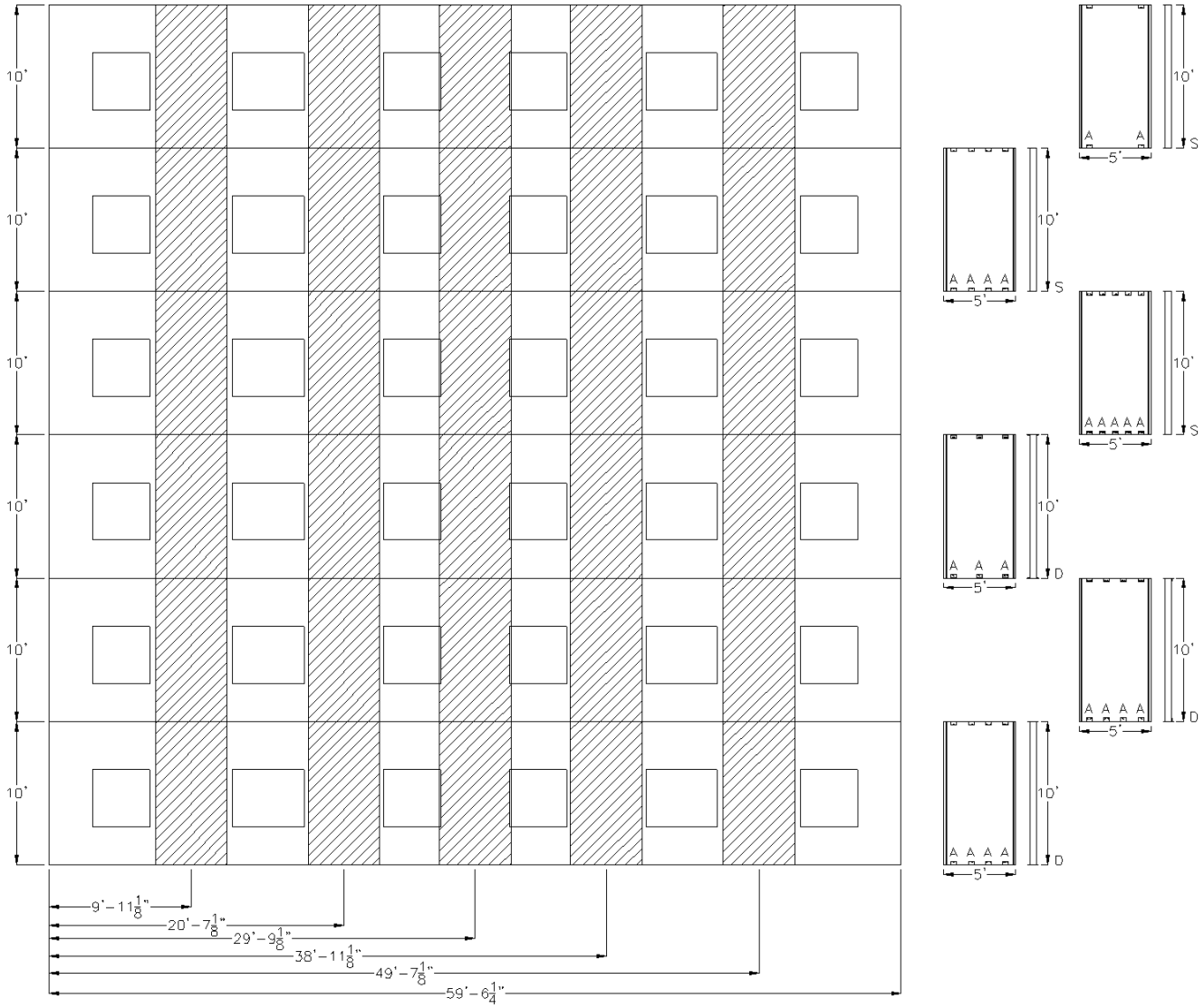


Figure B62. Archetype 88.

Table B59—Archetype 100 (4_E_6_2_ HR_HG_ DX_LP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.21	1.06	403.1	5.21	2.02	2.59
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.53	1.06	1010.6	5.21	5.05	1.03
4	2	M	2.5	6.875	5	A3	2	D	2084.0	4	7/8	0.46	1.79	1491.0	10.42	7.46	1.40
3	2	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/4	0.45	2.28	1846.0	15.63	9.23	1.69
2	2	M	2.5	9.625	7	A3	3	D	3126.0	4	1 3/8	0.46	2.52	2077.8	15.63	10.39	1.50
1	2	M	2.5	9.625	7	A3	3	D	3126.0	4	1 3/4	0.46	2.66	2190.7	15.63	10.95	1.43

* There are five of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for reinforced option and before applying C_d.

Table B60—Archetype 112 (4_E_6_2_ HR_LG_ DX_LP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.21	1.06	395.5	5.21	1.98	2.63
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.47	1.05	879.7	5.21	4.40	1.18
4	2	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.53	1.33	1262.6	7.82	6.31	1.24
3	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.47	1.84	1545.5	10.42	7.73	1.35
2	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.53	1.83	1730.2	10.42	8.65	1.20
1	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.56	1.79	1820.4	10.42	9.10	1.14

*There are five of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for reinforced option and before applying C_d.

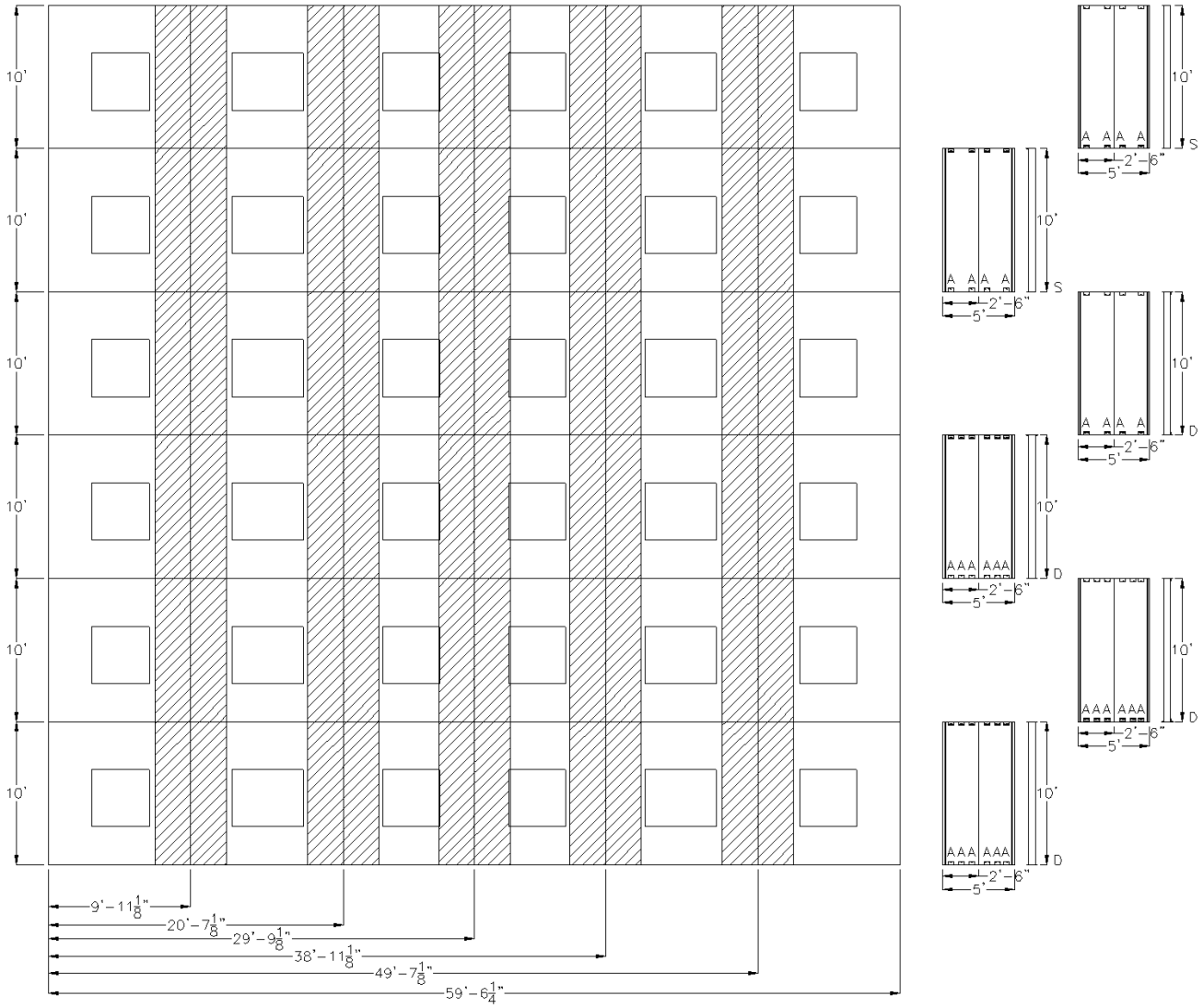


Figure B63. Archetype 100.

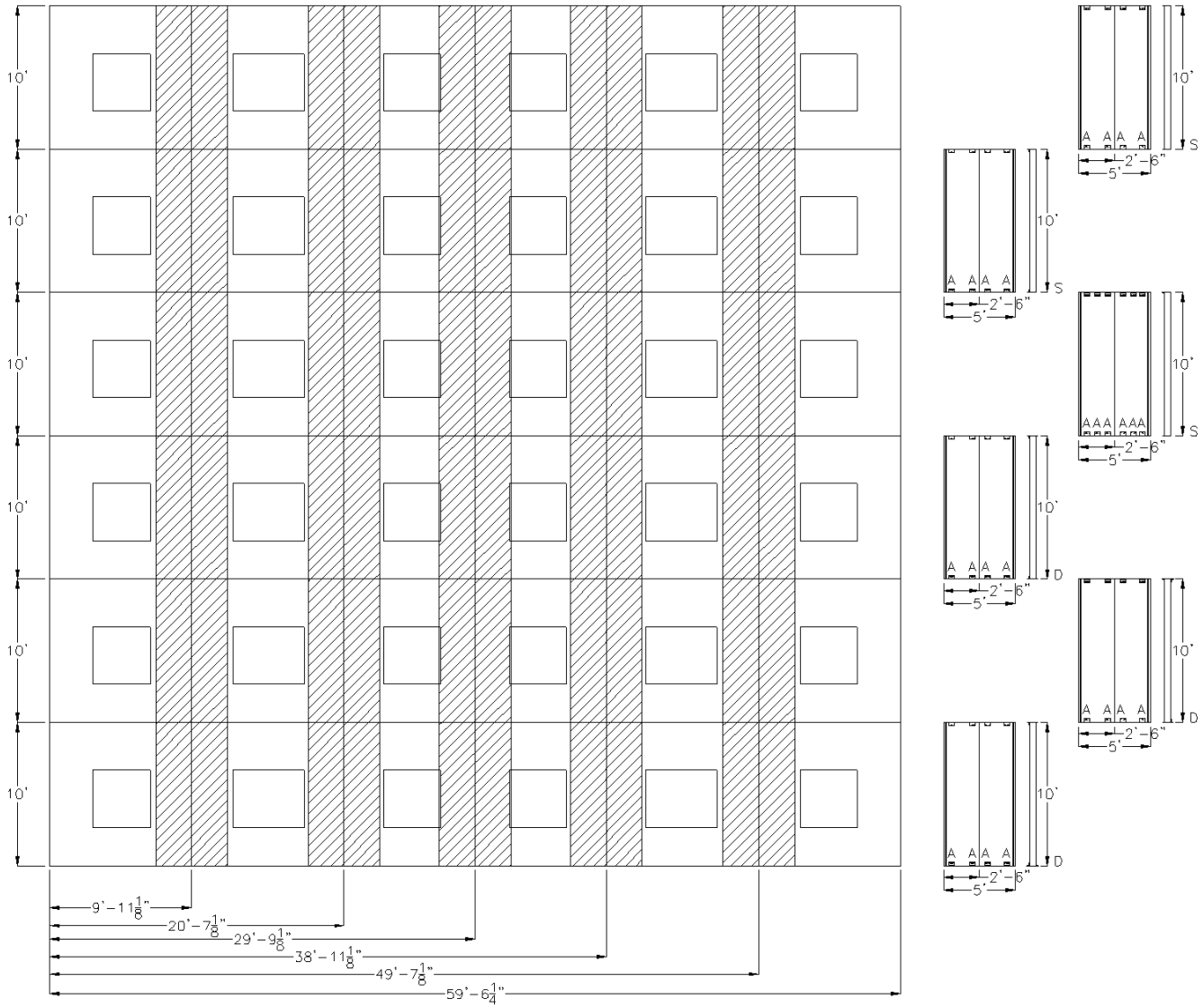


Figure B64. Archetype 112.

Table B61—Archetype 124 (4_E_6_2_ MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.52	0.30	277.2	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.63	0.42	468.7	-	-	-
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.78	0.52	734.5	-	-	-
3	2	M	2.5	6.875	5	A3	2	S	1042.0	4	7/8	0.77	0.57	789.9	-	-	-
2	2	M	2.5	9.625	7	A3	2	S	1042.0	4	1	0.77	0.62	860.1	-	-	-
1	2	M	2.5	9.625	7	A3	2	D	2084.0	4	1 1/2	0.80	1.06	1527.1	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.52	0.30	277.2	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.63	0.42	468.7	-	-	-
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.78	0.52	734.5	-	-	-
3	2	M	2.5	6.875	5	A3	2	S	1042.0	4	7/8	0.77	0.57	789.9	-	-	-
2	2	M	2.5	9.625	7	A3	2	S	1042.0	4	1	0.77	0.62	860.1	-	-	-
1	2	M	2.5	9.625	7	A3	2	D	2084.0	4	1 1/2	0.80	1.06	1527.1	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	S	5	4.125	3	A3	3	S	781.5	4	3/8	0.52	0.52	487.0	22.14	10.08	2.20
5	3	S	5	4.125	3	A3	6	S	1563.0	4	3/4	0.63	1.22	1371.9	33.87	25.27	1.34
4	3	S	5	4.125	3	A3	4	D	2084.0	4	1	0.78	1.42	1995.4	41.68	37.28	1.12
3	3	S	5	6.875	5	A3	6	D	3126.0	4	1 1/4	0.77	1.85	2550.1	57.31	46.15	1.24
2	3	S	5	9.625	7	A3	6	D	3126.0	4	1 1/2	0.77	2.08	2889.6	57.31	51.94	1.10
1	3	S	5	9.625	7	A3	6	D	3126.0	4	2	0.80	1.83	2633.2	67.73	54.77	1.24

*Values shown are for unreinforced option and before applying C_d.

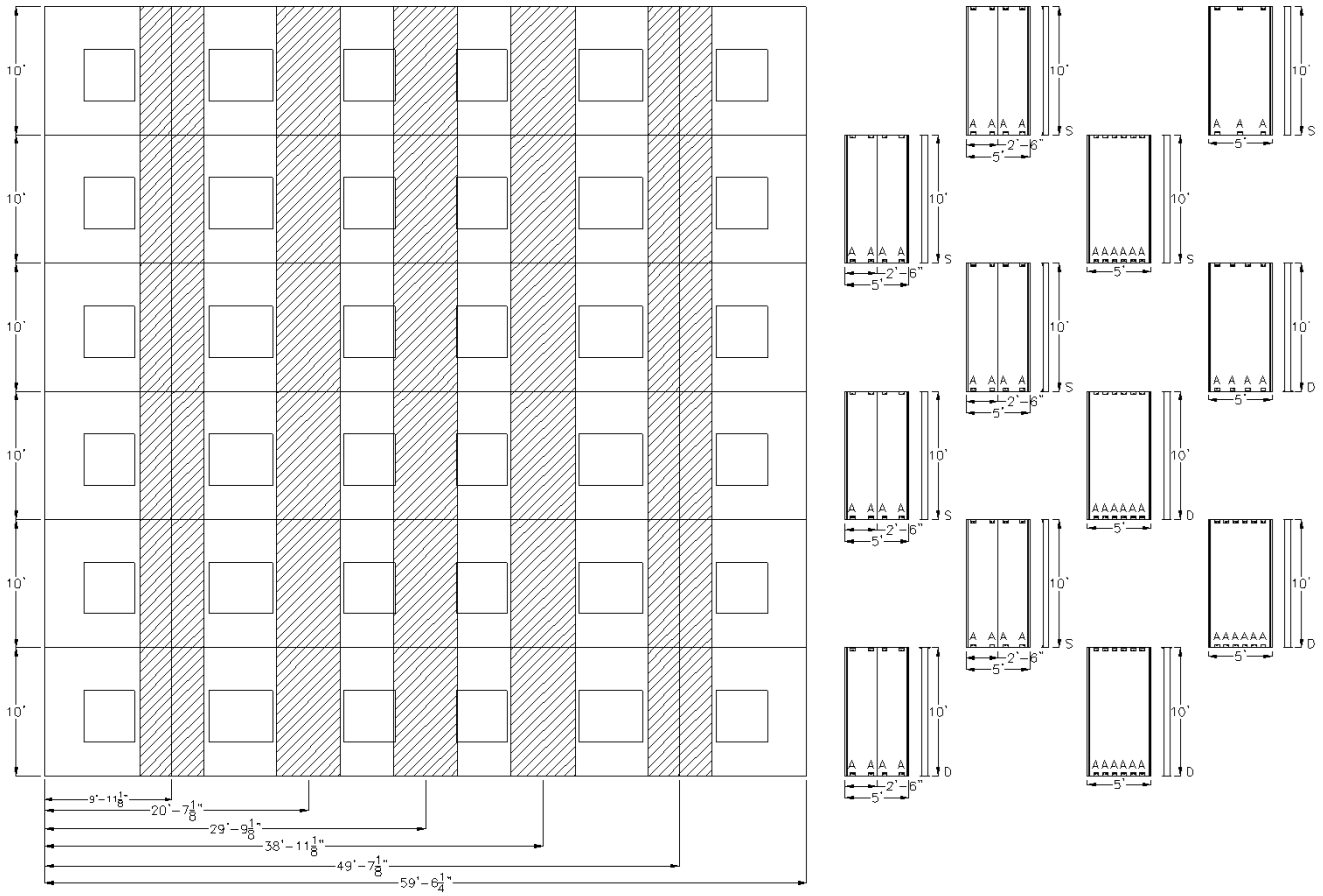


Figure B65. Archetype 124.

Table B62—Archetype 136 (4_E_6_2_ MR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.13	1.11	263.7	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.22	1.09	439.9	-	-	-
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.27	1.08	527.8	-	-	-
3	2	M	2.5	4.125	3	A3	2	S	1042.0	4	7/8	0.32	1.08	619.0	-	-	-
2	2	M	2.5	6.875	5	A3	2	S	1042.0	4	1	0.28	1.20	610.8	-	-	-
1	2	M	2.5	9.625	7	A3	2	S	1042.0	4	1 1/8	0.28	1.29	655.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.13	1.11	263.7	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.22	1.09	439.9	-	-	-
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.27	1.08	527.8	-	-	-
3	2	M	2.5	4.125	3	A3	2	S	1042.0	4	7/8	0.32	1.08	619.0	-	-	-
2	2	M	2.5	6.875	5	A3	2	S	1042.0	4	1	0.28	1.20	610.8	-	-	-
1	2	M	2.5	9.625	7	A3	2	S	1042.0	4	1 1/8	0.28	1.29	655.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	S	5	4.125	3	A3	3	S	781.5	4	3/8	0.13	2.03	483.4	22.14	9.89	2.24
5	3	S	5	4.125	3	A3	5	S	1302.5	4	5/8	0.22	2.91	1172.9	29.96	21.99	1.36
4	3	S	5	4.125	3	A3	4	D	2084.0	4	7/8	0.27	3.58	1752.5	41.68	31.57	1.32
3	3	S	5	4.125	3	A3	5	D	2605.0	4	1 1/8	0.32	3.77	2163.2	49.50	38.64	1.28
2	3	S	5	6.875	5	A3	5	D	2605.0	4	1 3/8	0.28	4.88	2476.5	49.50	43.26	1.14
1	3	S	5	9.625	7	A3	5	D	2605.0	4	1 1/2	0.28	5.13	2597.1	49.50	45.51	1.09

*Values shown are for reinforced option and before applying C_d.

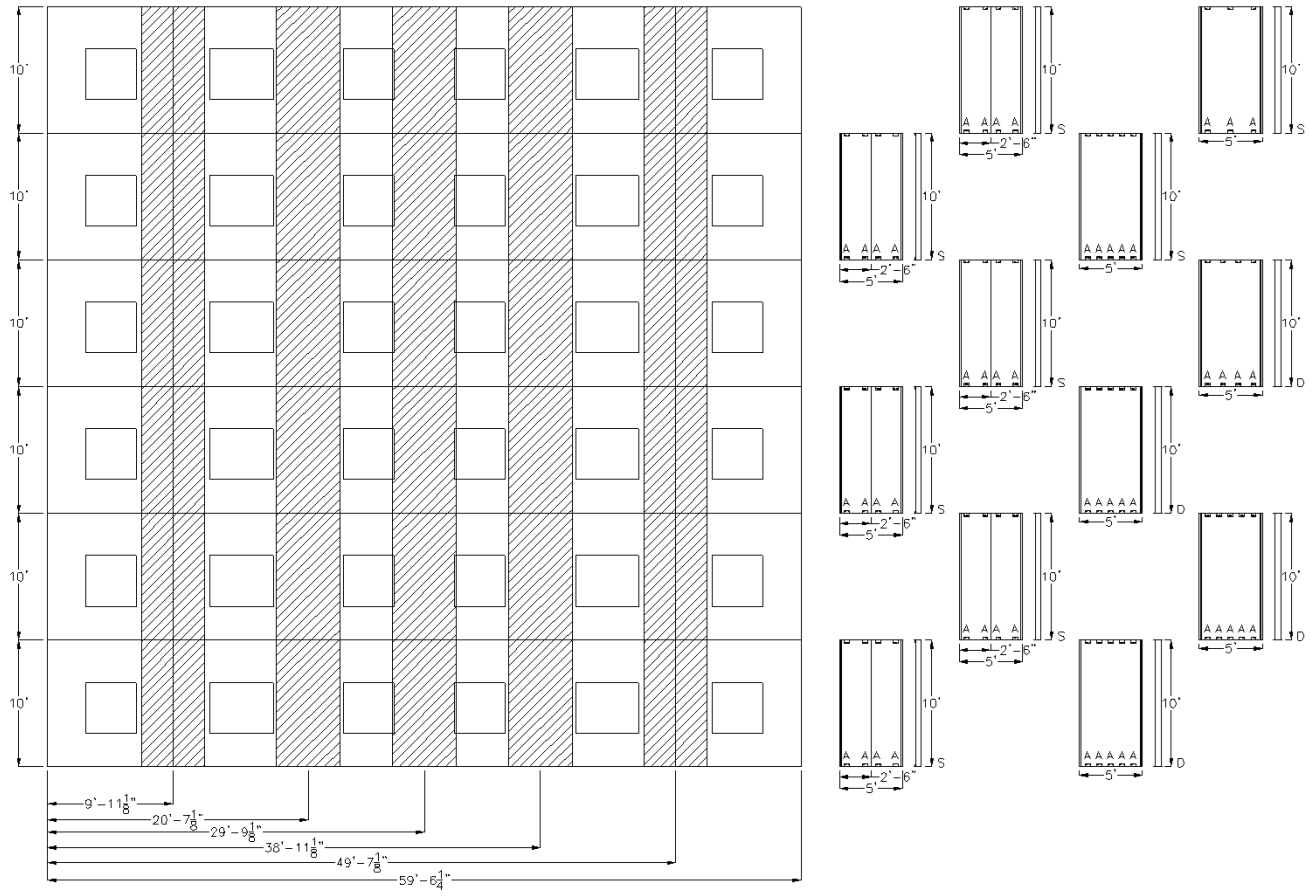


Figure B66. Archetype 136.

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	1,055	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I=	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t (h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.2485					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	42,708					

DESIGN SUMMARY

Total base shear V = 262.14

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	125	9,280	0.217	57.0		
	10.00						57.0	
5		50.0	186	11,359	0.266	69.7		570
	10.00						126.7	
4		40.0	186	8,983	0.210	55.1		1,836
	10.00						181.8	
3		30.0	186	6,637	0.155	40.7		3,655
	10.00						222.6	
2		20.0	186	4,333	0.101	26.6		5,880
	10.00						249.2	
1		10.0	188	2,116	0.050	13.0		8,372
	10.00						262.1	
		0.0						10,993

Seismic base shear calculation, low gravity, Index Building 4, 6-story, R = 4

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	1,270	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I=	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t (h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.2485					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	50,434					

DESIGN SUMMARY

Total base shear V = 315.47

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	125	9,280	0.184	58.0		
	10.00						58.0	
5		50.0	228	13,986	0.277	87.5		580
	10.00						145.5	
4		40.0	228	11,060	0.219	69.2		2,036
	10.00						214.7	
3		30.0	228	8,173	0.162	51.1		4,183
	10.00						265.8	
2		20.0	228	5,335	0.106	33.4		6,841
	10.00						299.2	
1		10.0	231	2,600	0.052	16.3		9,833
	10.00						315.5	
		0.0						12,988

Seismic base shear calculation, high gravity, Index Building 4, 6-story, R = 4

Table B63—Tributary load calculation, Index Building 4, 6-story, N-S direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	1	205.0	0.086	56.96	4.93	58.05	5.02
	2	392.0	0.165		9.42		9.60
	3	**392.0**	**0.165**		**9.42**		**9.60**
	4	392.0	0.165		9.42		9.60
	5	392.0	0.165		9.42		9.60
	6	392.0	0.165		9.42		9.60
	7	205.0	0.086		4.93		5.02
Story 5	1	205.0	0.086	126.68	10.96	145.53	12.59
	2	392.0	0.165		20.95		24.07
	3	**392.0**	**0.165**		**20.95**		**24.07**
	4	392.0	0.165		20.95		24.07
	5	392.0	0.165		20.95		24.07
	6	392.0	0.165		20.95		24.07
	7	205.0	0.086		10.96		12.59
Story 4	1	205.0	0.086	181.82	15.73	214.71	18.57
	2	392.0	0.165		30.07		35.51
	3	**392.0**	**0.165**		**30.07**		**35.51**
	4	392.0	0.165		30.07		35.51
	5	392.0	0.165		30.07		35.51
	6	392.0	0.165		30.07		35.51
	7	205.0	0.086		15.73		18.57
Story 3	1	205.0	0.086	222.56	19.25	265.83	22.99
	2	392.0	0.165		36.81		43.97
	3	**392.0**	**0.165**		**36.81**		**43.97**
	4	392.0	0.165		36.81		43.97
	5	392.0	0.165		36.81		43.97
	6	392.0	0.165		36.81		43.97
	7	205.0	0.086		19.25		22.99
Story 2	1	205.0	0.086	249.15	21.55	299.20	25.88
	2	392.0	0.165		41.21		49.49
	3	**392.0**	**0.165**		**41.21**		**49.49**
	4	392.0	0.165		41.21		49.49
	5	392.0	0.165		41.21		49.49
	6	392.0	0.165		41.21		49.49
	7	205.0	0.086		21.55		25.88
Story 1	1	205.0	0.086	262.14	22.67	315.47	27.29
	2	392.0	0.165		43.36		52.18
	3	**392.0**	**0.165**		**43.36**		**52.18**
	4	392.0	0.165		43.36		52.18
	5	392.0	0.165		43.36		52.18
	6	392.0	0.165		43.36		52.18
	7	205.0	0.086		22.67		27.29

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B64—Tributary load calculation, Index Building 4, 6-story, E-W direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	312.0	0.135	56.96	7.71	58.05	7.86
	B	**340.0**	**0.148**		**8.41**		**8.57**
	C	271.0	0.118		6.70		6.83
	D	781.0	0.339		19.31		19.68
	E	**600.0**	**0.260**		**14.83**		**15.12**
Story 5	A	312.0	0.135	126.68	17.15	145.53	19.71
	B	**340.0**	**0.148**		**18.69**		**21.48**
	C	271.0	0.118		14.90		17.12
	D	781.0	0.339		42.94		49.33
	E	**600.0**	**0.260**		**32.99**		**37.90**
Story 4	A	312.0	0.135	181.82	24.62	214.71	29.08
	B	**340.0**	**0.148**		**26.83**		**31.68**
	C	271.0	0.118		21.39		25.25
	D	781.0	0.339		61.63		72.78
	E	**600.0**	**0.260**		**47.35**		**55.91**
Story 3	A	312.0	0.135	222.56	30.14	265.83	36.00
	B	**340.0**	**0.148**		**32.84**		**39.23**
	C	271.0	0.118		26.18		31.27
	D	781.0	0.339		75.44		90.11
	E	**600.0**	**0.260**		**57.96**		**69.23**
Story 2	A	312.0	0.135	249.15	33.74	299.20	40.52
	B	**340.0**	**0.148**		**36.77**		**44.15**
	C	271.0	0.118		29.31		35.19
	D	781.0	0.339		84.46		101.42
	E	**600.0**	**0.260**		**64.88**		**77.92**
Story 1	A	312.0	0.135	262.14	35.50	315.47	42.72
	B	**340.0**	**0.148**		**38.68**		**46.55**
	C	271.0	0.118		30.83		37.11
	D	781.0	0.339		88.86		106.94
	E	**600.0**	**0.260**		**68.27**		**82.15**

* Seismic load for shear wall line E is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B65—Archetype 28 (4_3_ 6_1_HR_HG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.20	1.63	397.2	-	-	-
5	3	M	2.5	4.125	3	A3	3	S	1563.0	4	1/2	0.50	1.78	1074.9	-	-	-
4	3	M	2.5	4.125	3	A3	2	D	2084.0	4	3/4	0.63	2.24	1693.4	-	-	-
3	3	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.72	2.19	1881.1	-	-	-
2	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1	0.63	3.10	2360.5	-	-	-
1	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/4	0.66	3.30	2625.1	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.20	1.00	364.3	13.03	4.80	2.71
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.50	0.88	794.7	16.93	12.04	1.41
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.63	0.89	1011.3	20.84	17.76	1.17
3	2	M	2.5	4.125	3	A3	2	D	2084.0	4	3/4	0.72	1.22	1575.2	26.05	21.98	1.18
2	2	M	2.5	6.875	5	A3	3	S	1563.0	4	7/8	0.63	1.23	1408.1	31.26	24.74	1.26
1	2	M	2.5	6.875	5	A3	3	S	1563.0	4	7/8	0.66	1.07	1280.1	31.26	26.09	1.20

*Values shown are for reinforced option and before applying C_d.

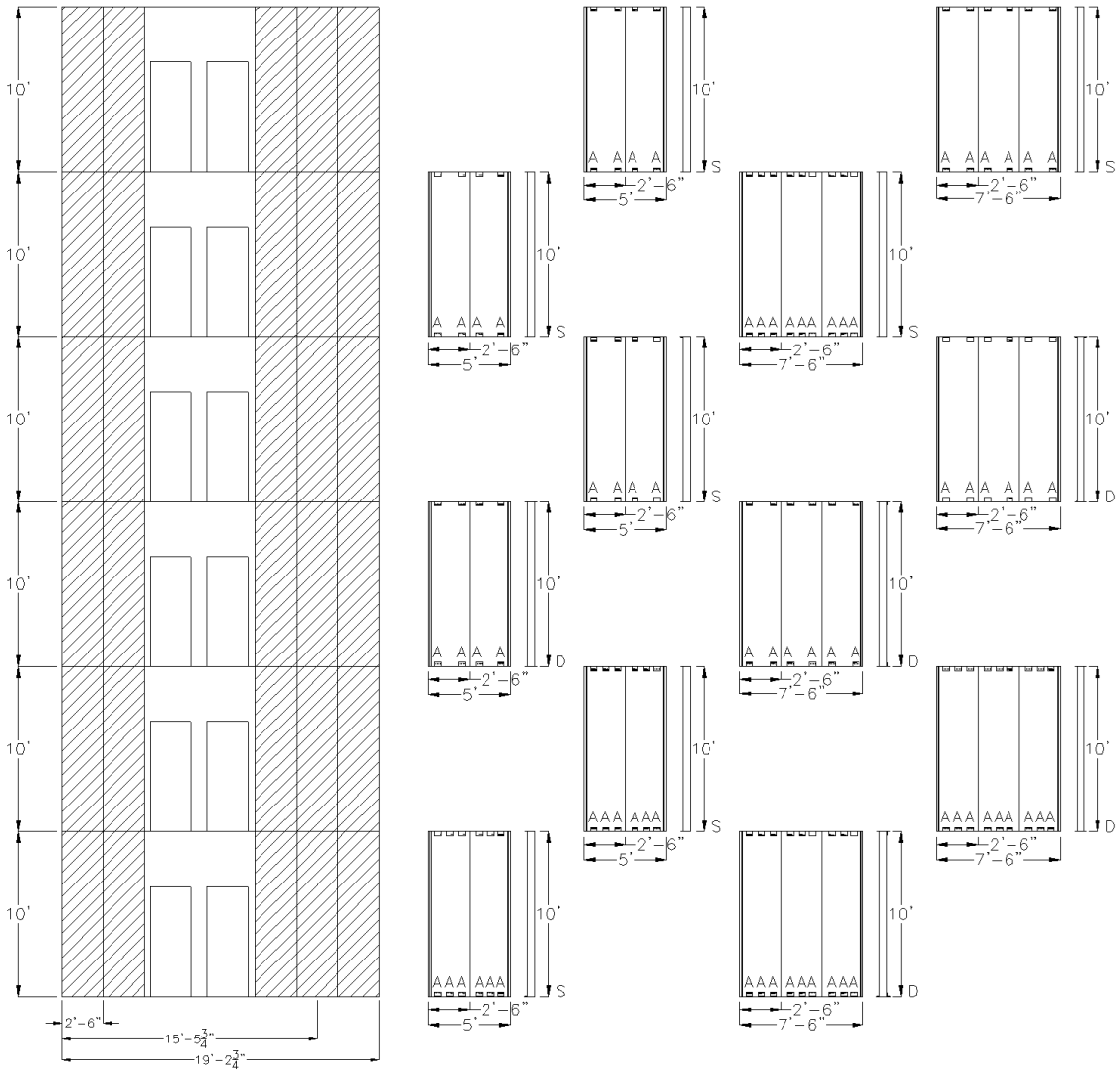


Figure B67. Archetype 28.

Table B66—Archetype 40 (4_3_ 6_1_HR_LG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.20	1.63	389.7	-	-	-
5	3	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.44	1.64	866.4	-	-	-
4	3	M	2.5	4.125	3	A3	3	S	1563.0	4	5/8	0.62	1.83	1350.2	-	-	-
3	3	M	2.5	4.125	3	A3	2	D	2084.0	4	3/4	0.72	2.07	1796.6	-	-	-
2	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1	0.57	3.14	2160.9	-	-	-
1	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/8	0.62	3.12	2322.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.20	1.00	357.6	13.03	4.71	2.77
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.44	1.01	795.7	13.03	10.48	1.24
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.62	0.89	981.9	16.93	15.04	1.13
3	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.72	0.76	986.2	20.84	18.41	1.13
2	2	M	2.5	6.875	5	A3	2	S	1042.0	4	3/4	0.57	0.85	879.6	28.66	20.60	1.39
1	2	M	2.5	6.875	5	A3	2	S	1042.0	4	3/4	0.62	0.76	851.9	28.66	21.68	1.32

*Values shown are for unreinforced option and before applying C_d.

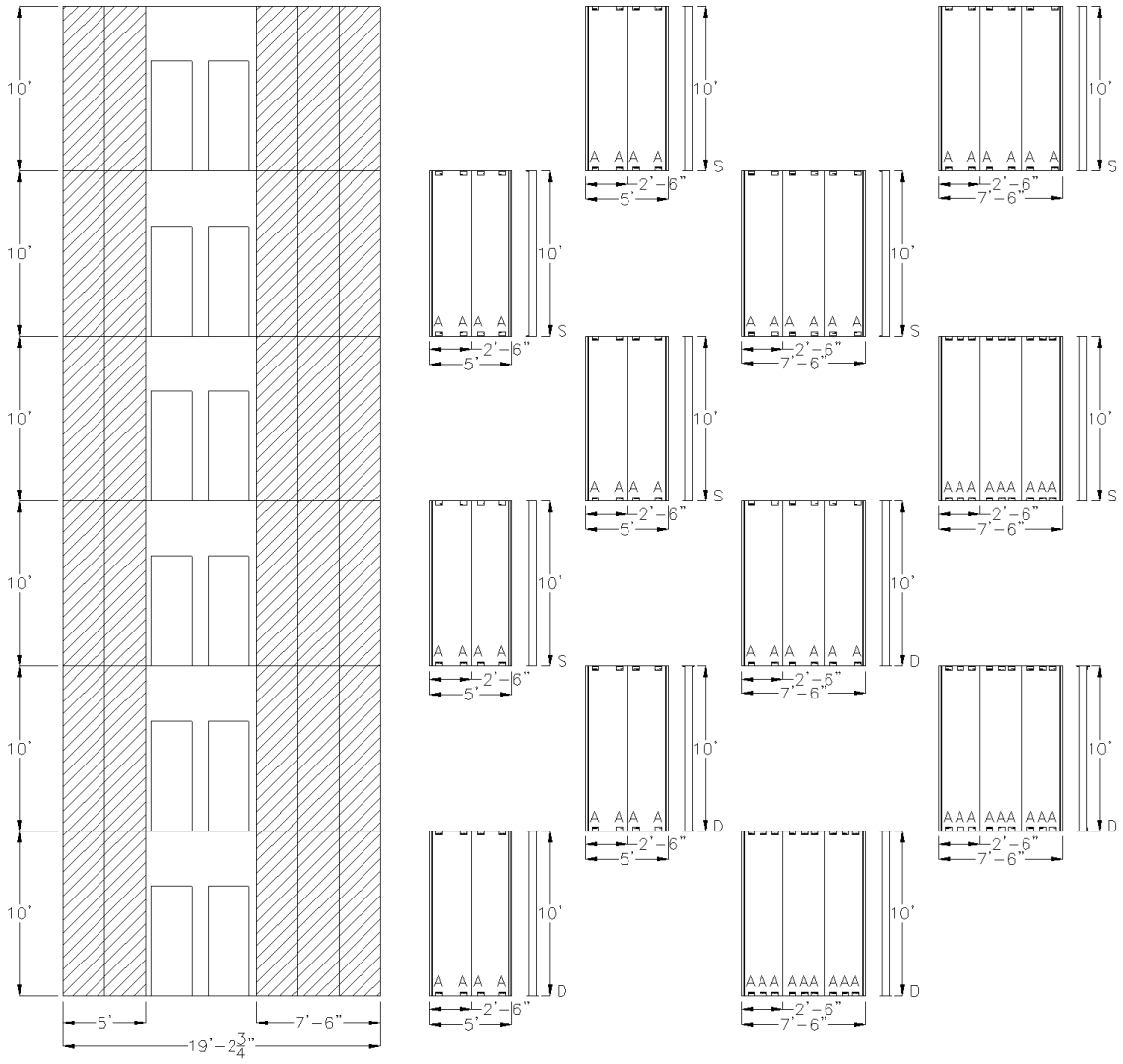


Figure B68. Archetype 40.

Table B67—Archetype 100 (4_E_6_2_ HR_HG_ DX_LP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.15	1.09	302.3	5.21	1.51	3.45
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.39	1.09	758.0	5.21	3.79	1.37
4	2	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.45	1.37	1118.3	7.82	5.59	1.40
3	2	M	2.5	6.875	5	A3	3	S	1563.0	4	1	0.49	1.58	1384.5	7.82	6.92	1.13
2	2	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/8	0.56	1.55	1558.3	7.82	7.79	1.00
1	2	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.50	1.83	1643.1	10.42	8.22	1.27

* There are five of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for reinforced option and before applying C_d.

Table B68—Archetype 112 (4_E_6_2_ HR_LG_ DX_LP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.15	1.09	296.7	5.21	1.48	3.51
5	2	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.34	1.08	659.8	5.21	3.30	1.58
4	2	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.49	1.06	947.0	5.21	4.73	1.10
3	2	M	2.5	4.125	3	A3	3	S	1563.0	4	1	0.48	1.33	1159.1	7.82	5.80	1.35
2	2	M	2.5	4.125	3	A3	3	S	1563.0	4	1 1/8	0.56	1.30	1297.7	7.82	6.49	1.20
1	2	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/4	0.49	1.54	1365.3	7.82	6.83	1.14

* There are five of (2)2.5ft walls along the wall line and all take equal load.
**Values shown are for reinforced option and before applying C_d.

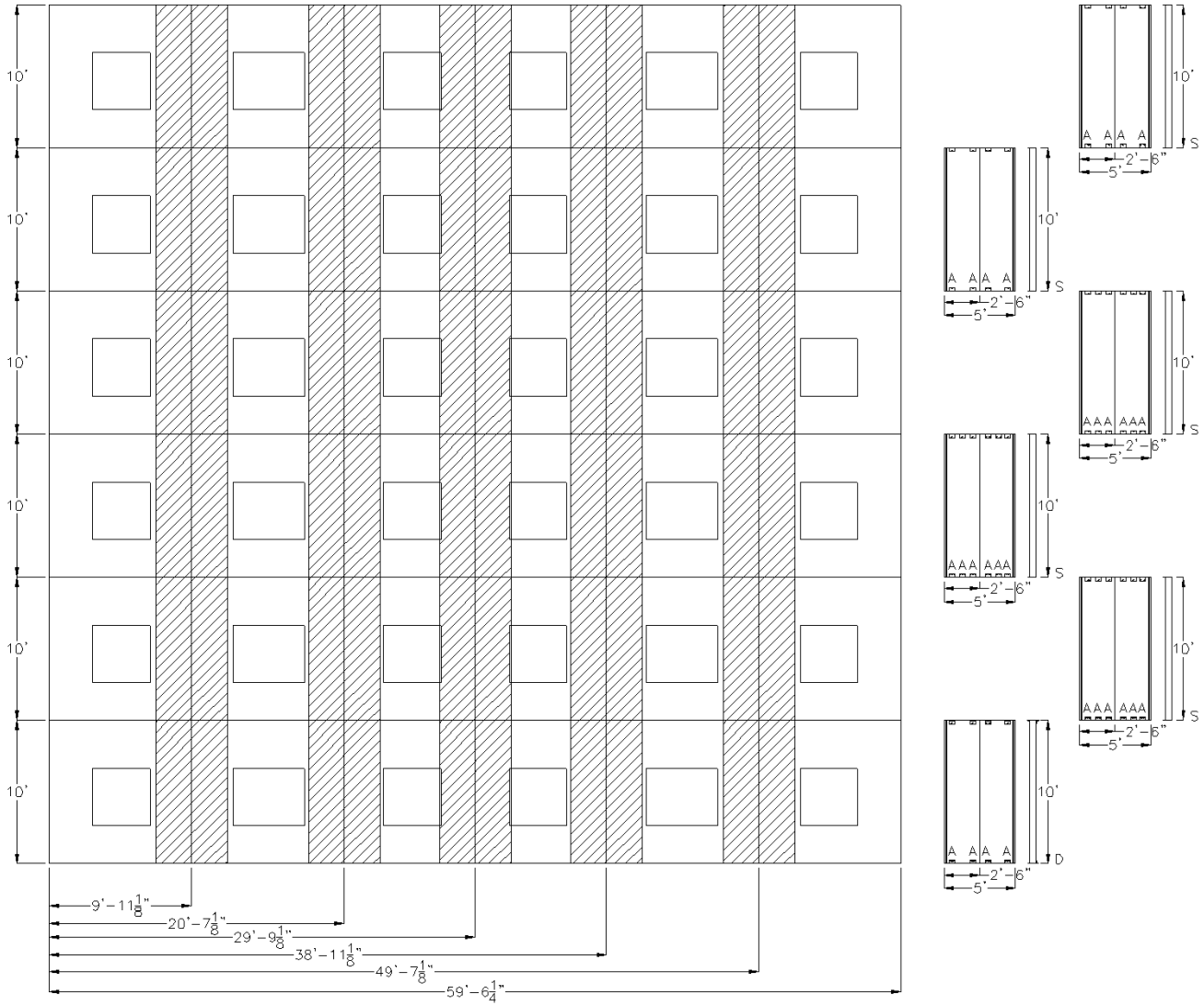


Figure B69. Archetype 100.

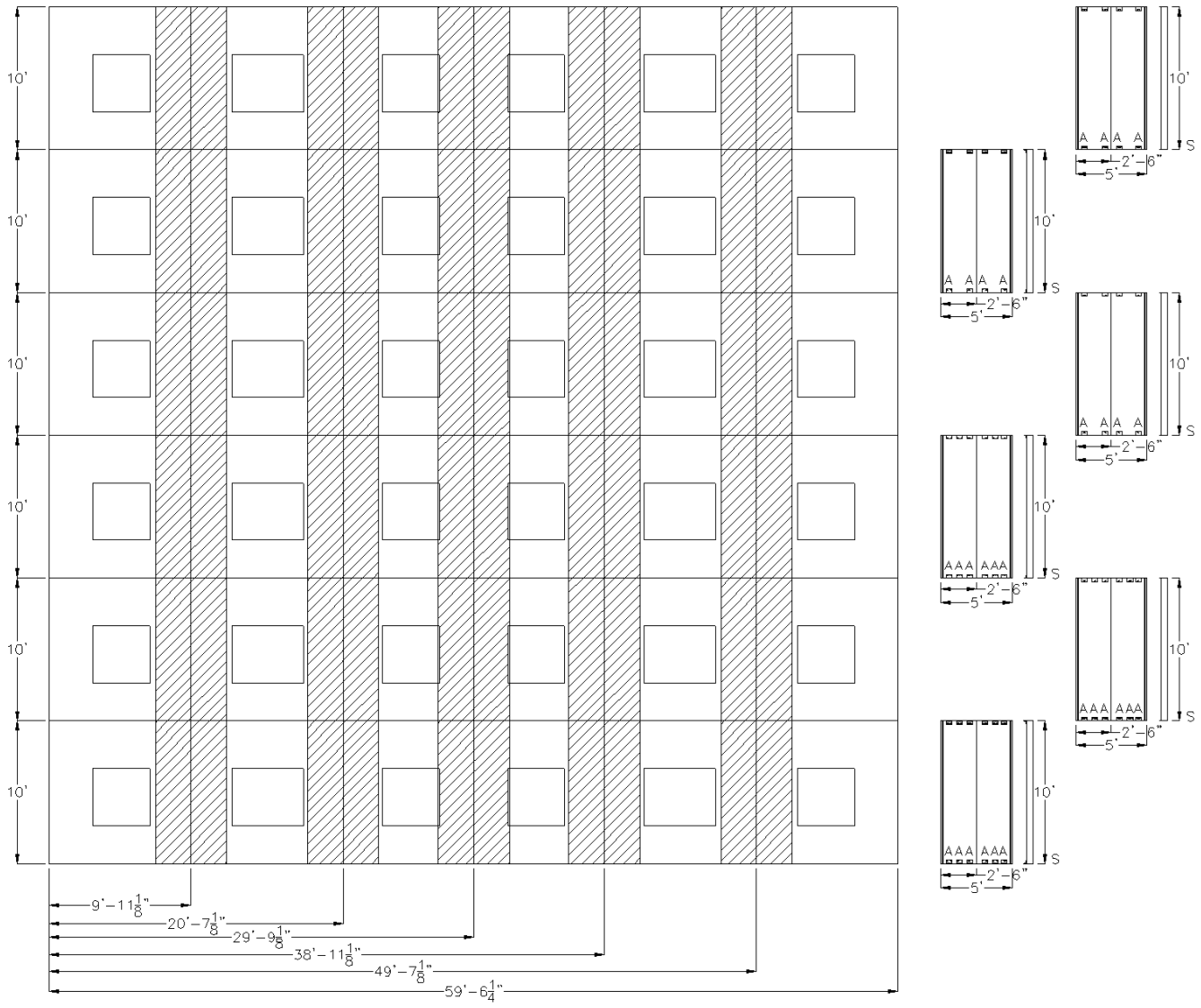


Figure B70. Archetype 112.

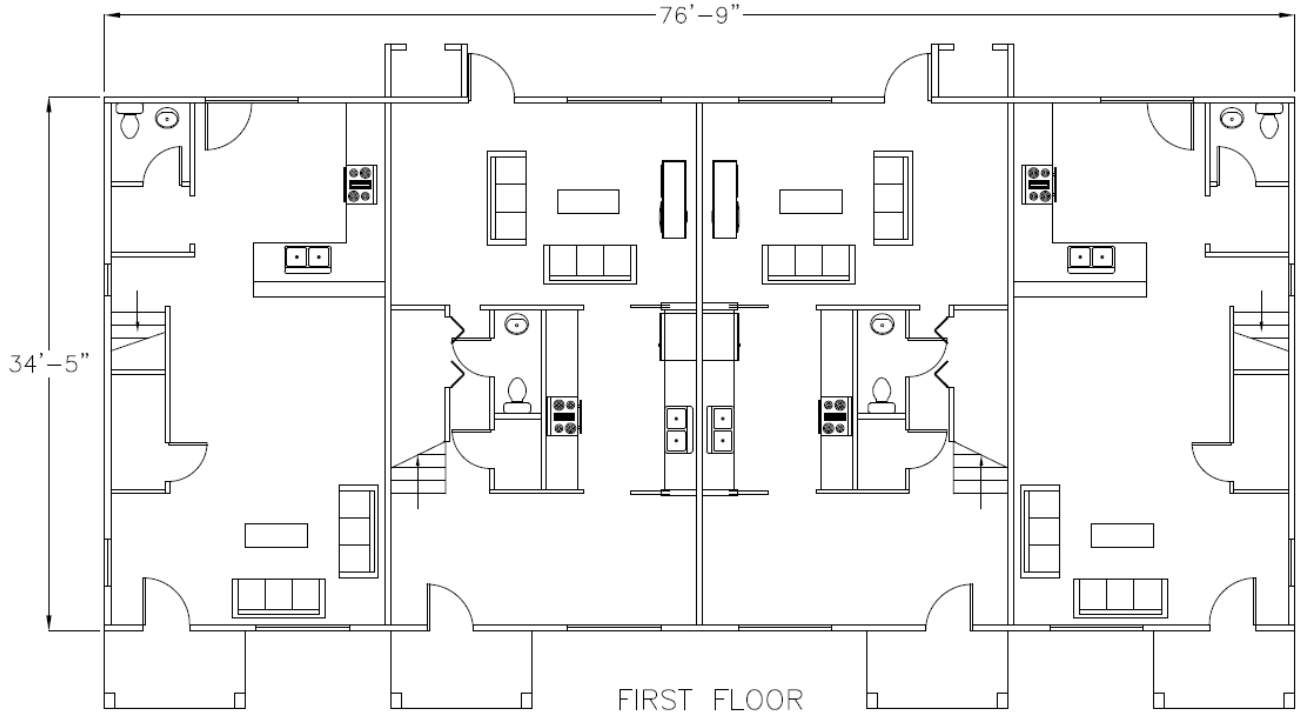


Figure B71. Index Building 5 first floor plan.

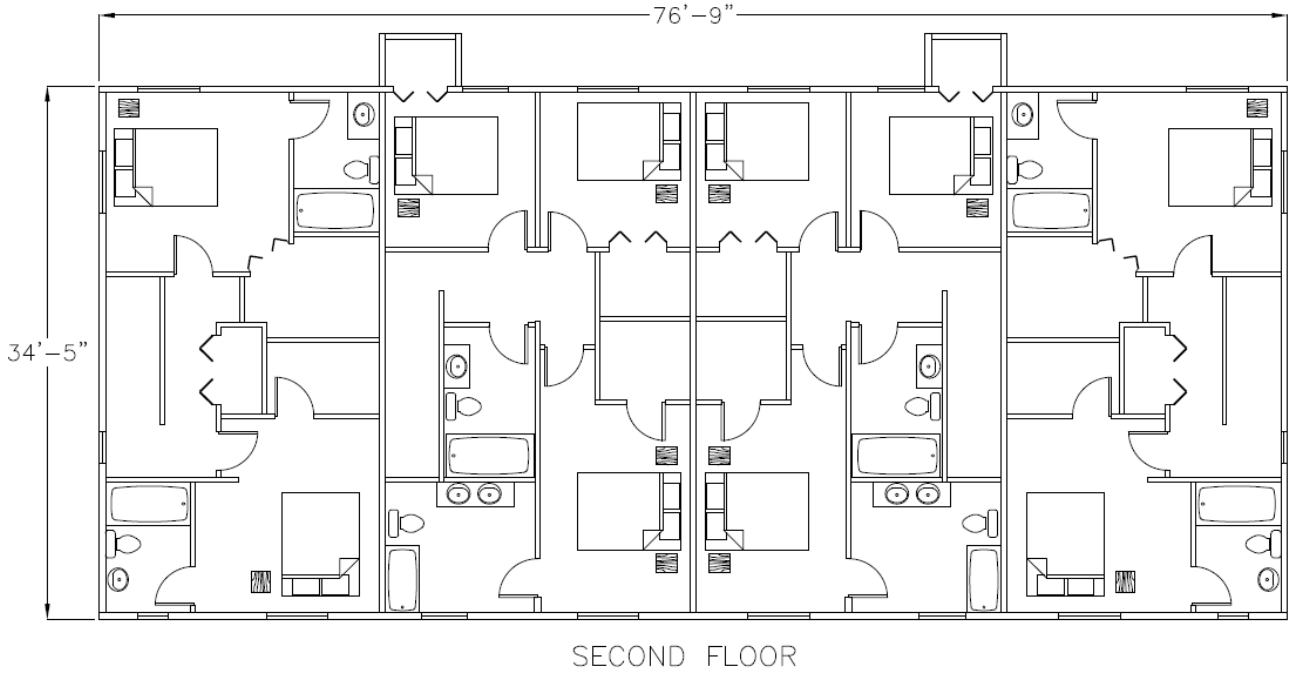


Figure B72. Index Building 5 second floor plan.

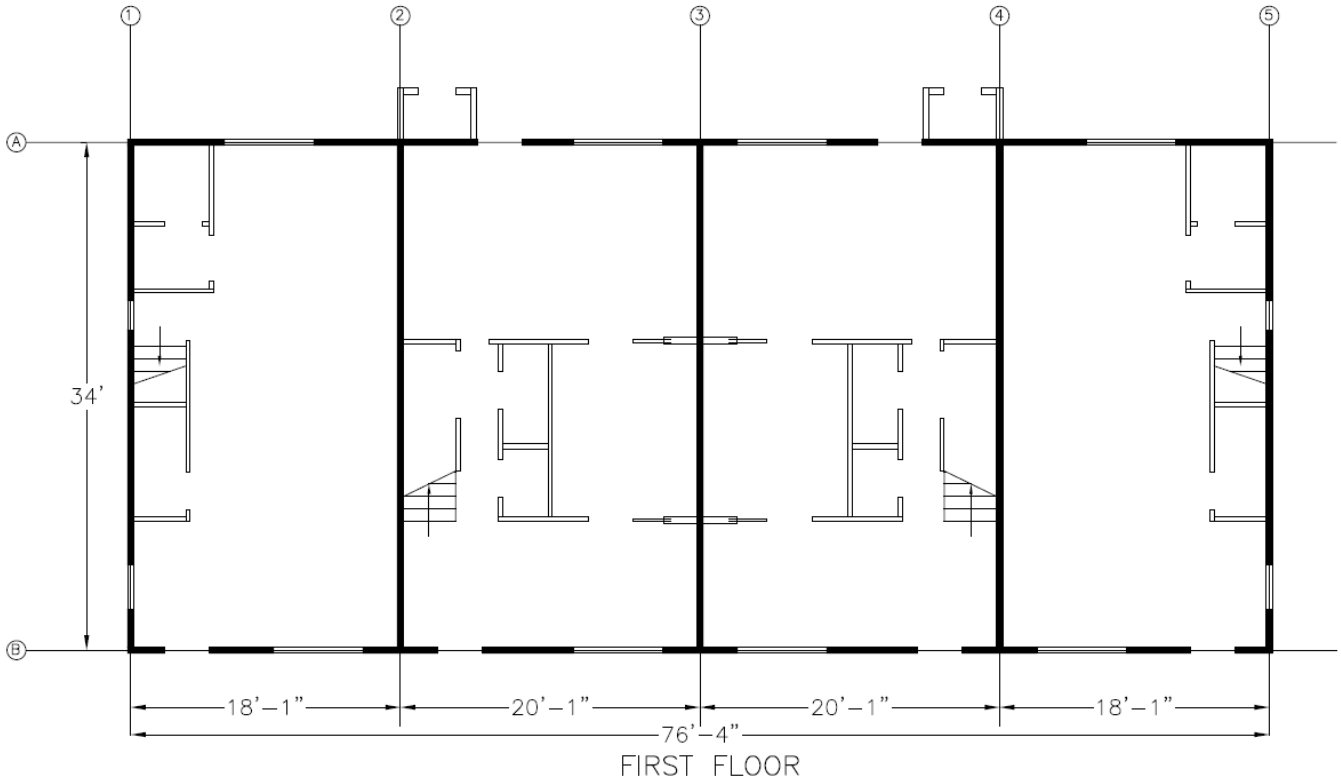


Figure B73. Index Building 5 first floor assigned shear walls.

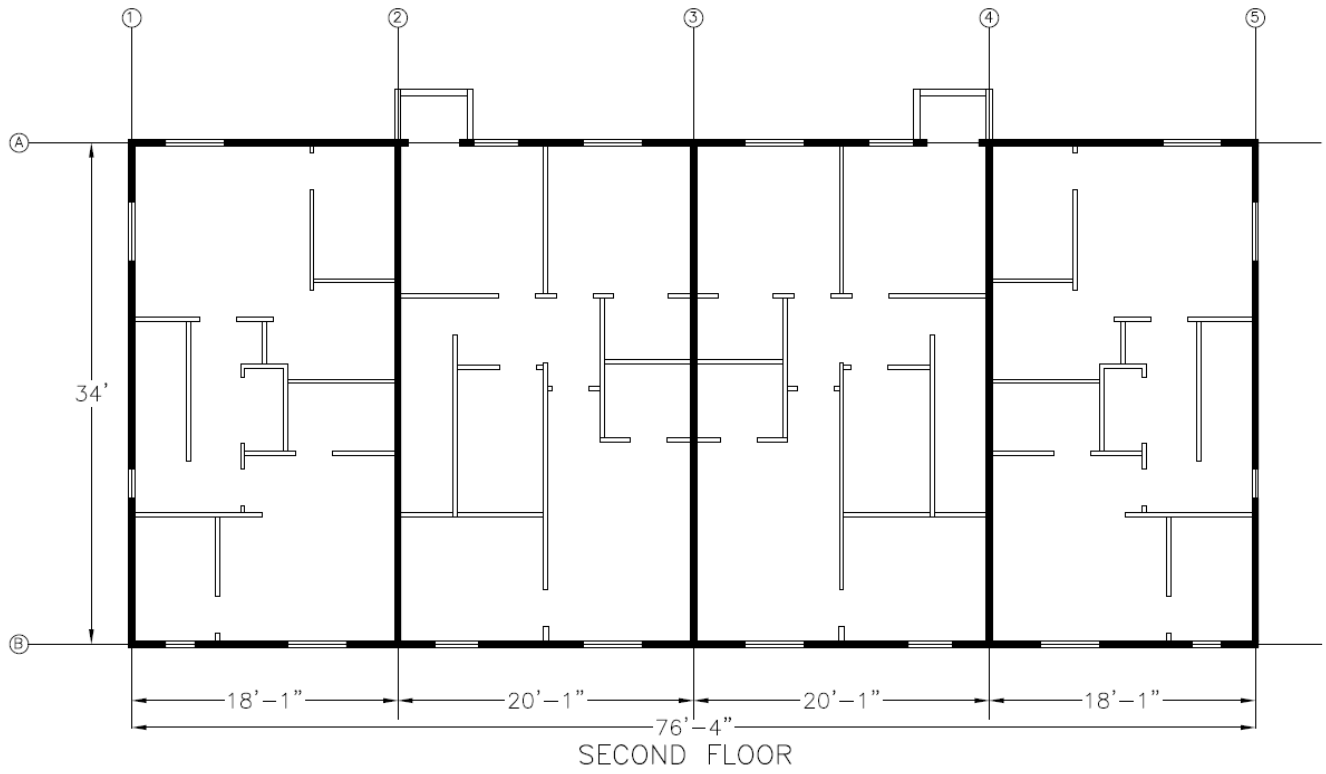


Figure B74. Index Building 5 second floor assigned shear walls.

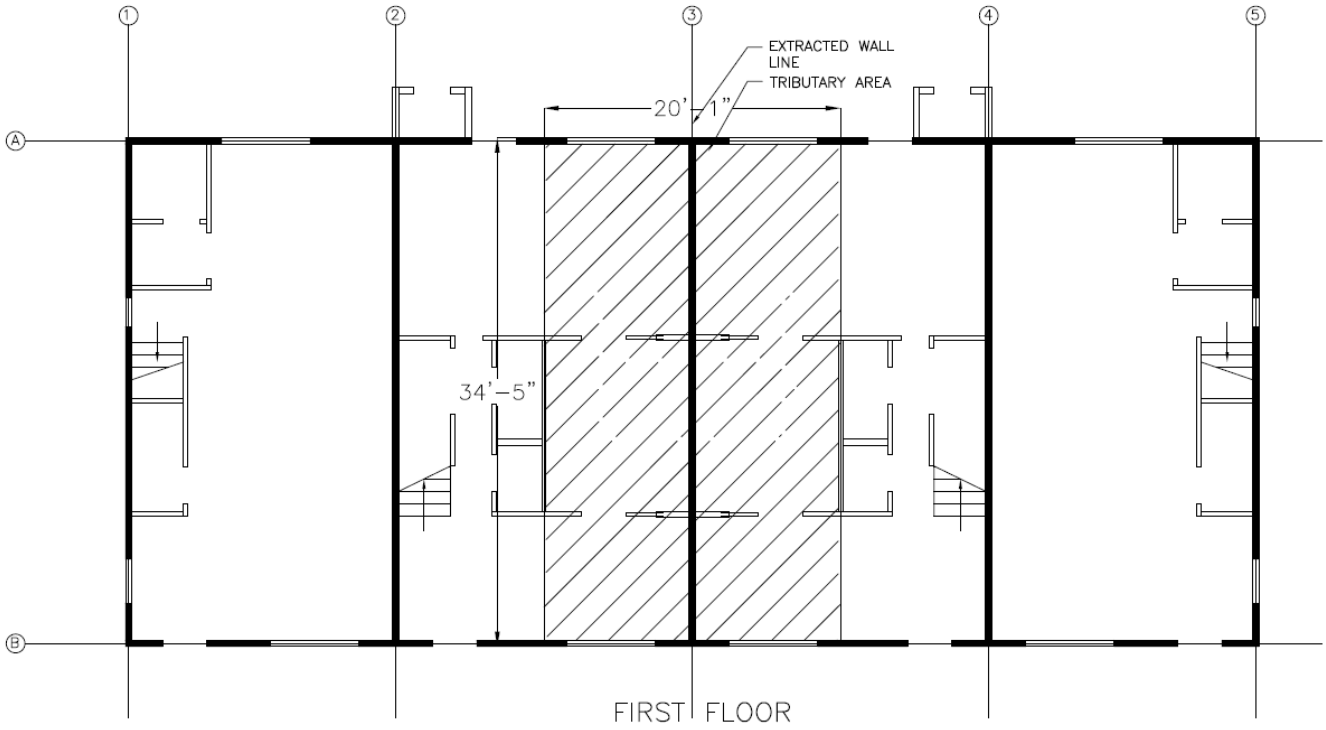


Figure B75. Index Building 5 first floor tributary area.

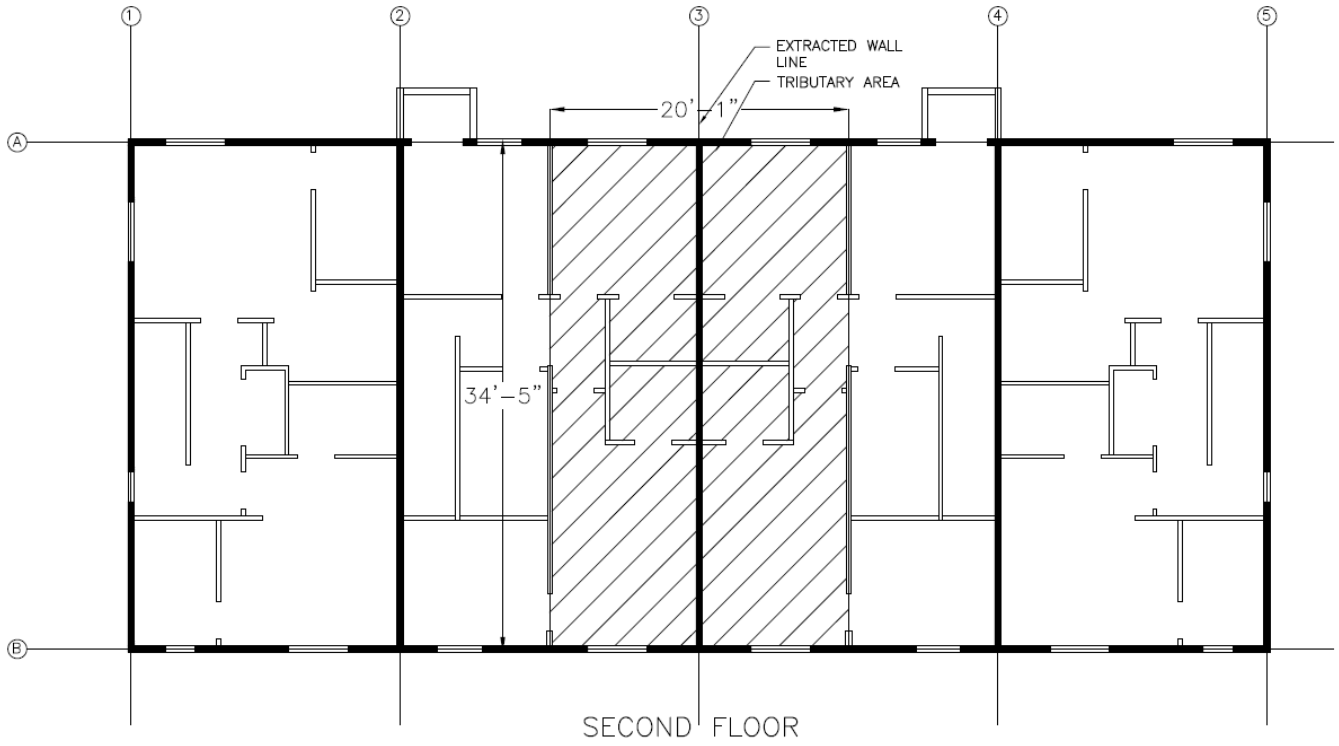


Figure B76. Index Building 5 second floor tributary area.

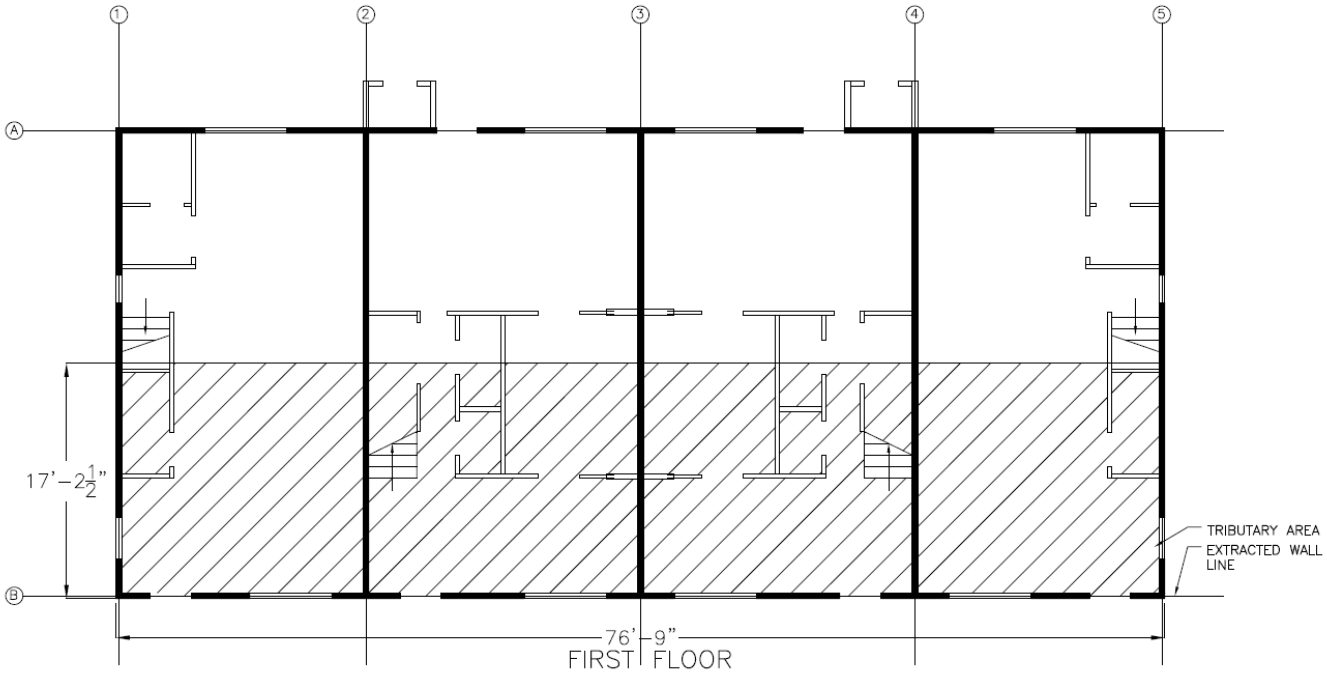


Figure B77. Index Building 5 first floor tributary area.

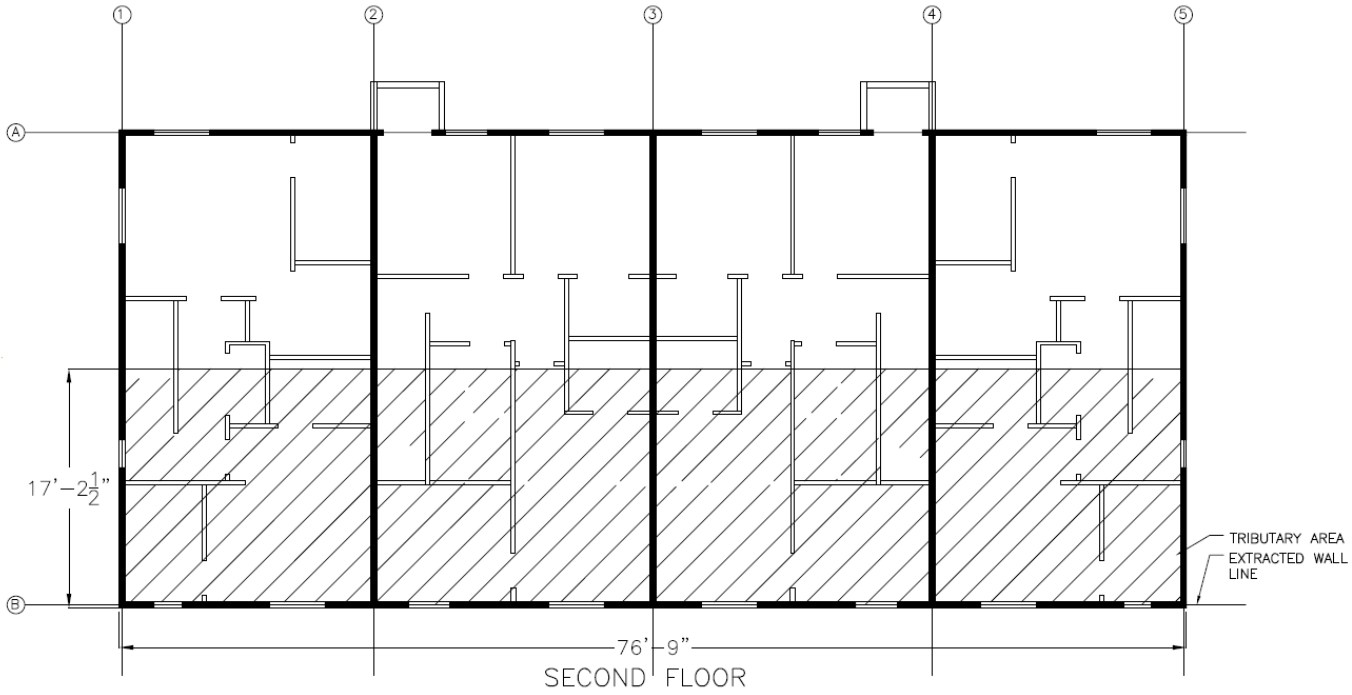


Figure B78. Index Building 5 second floor tributary area.

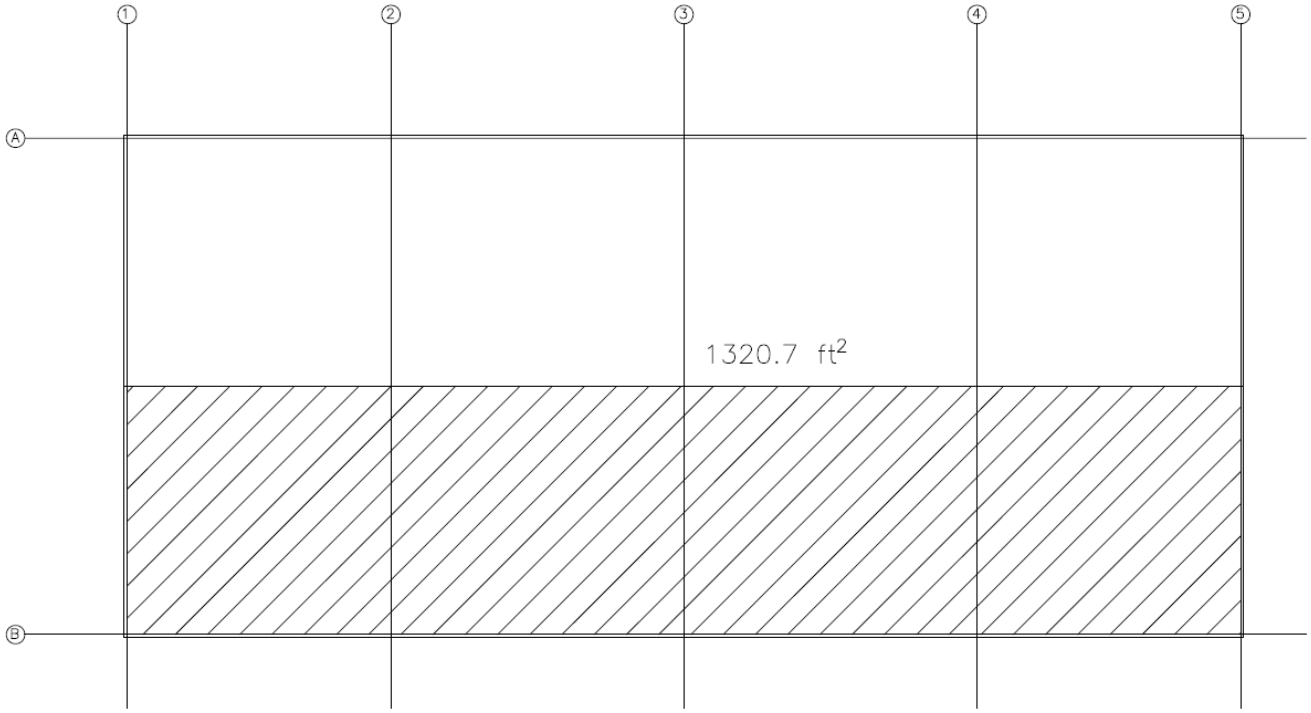


Figure B79. Index Building 5 tributary area.

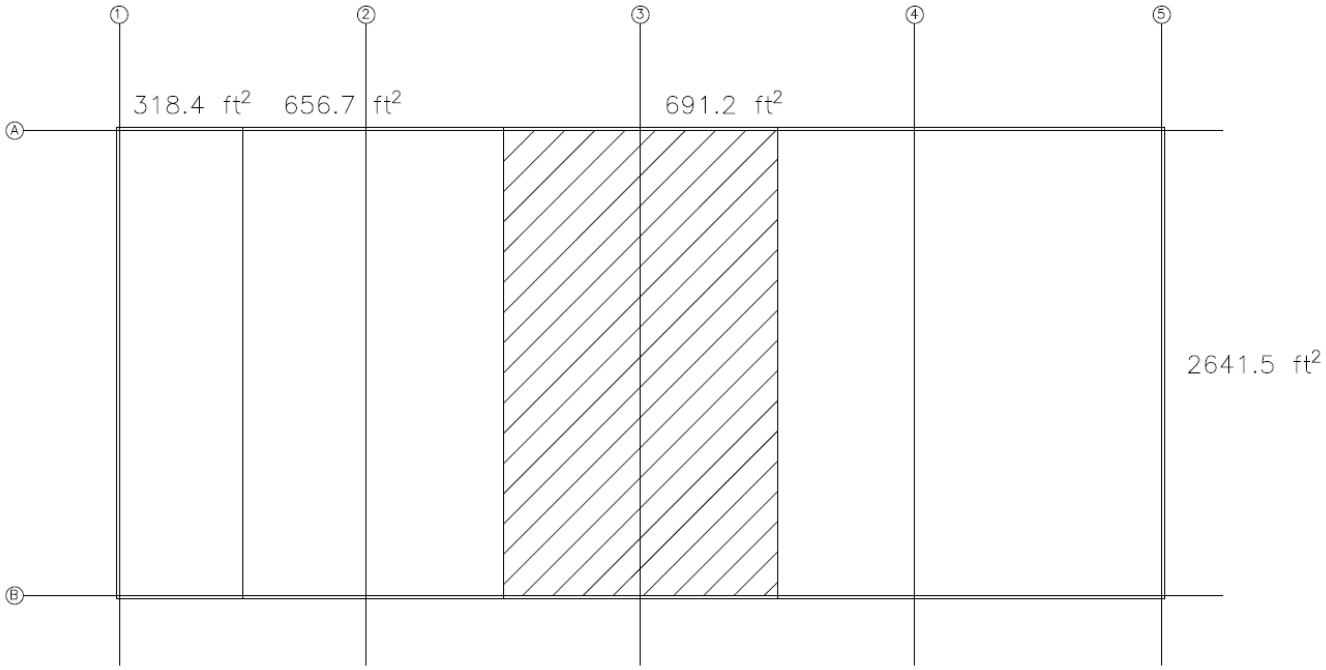


Figure B80. Index Building 5 tributary area.

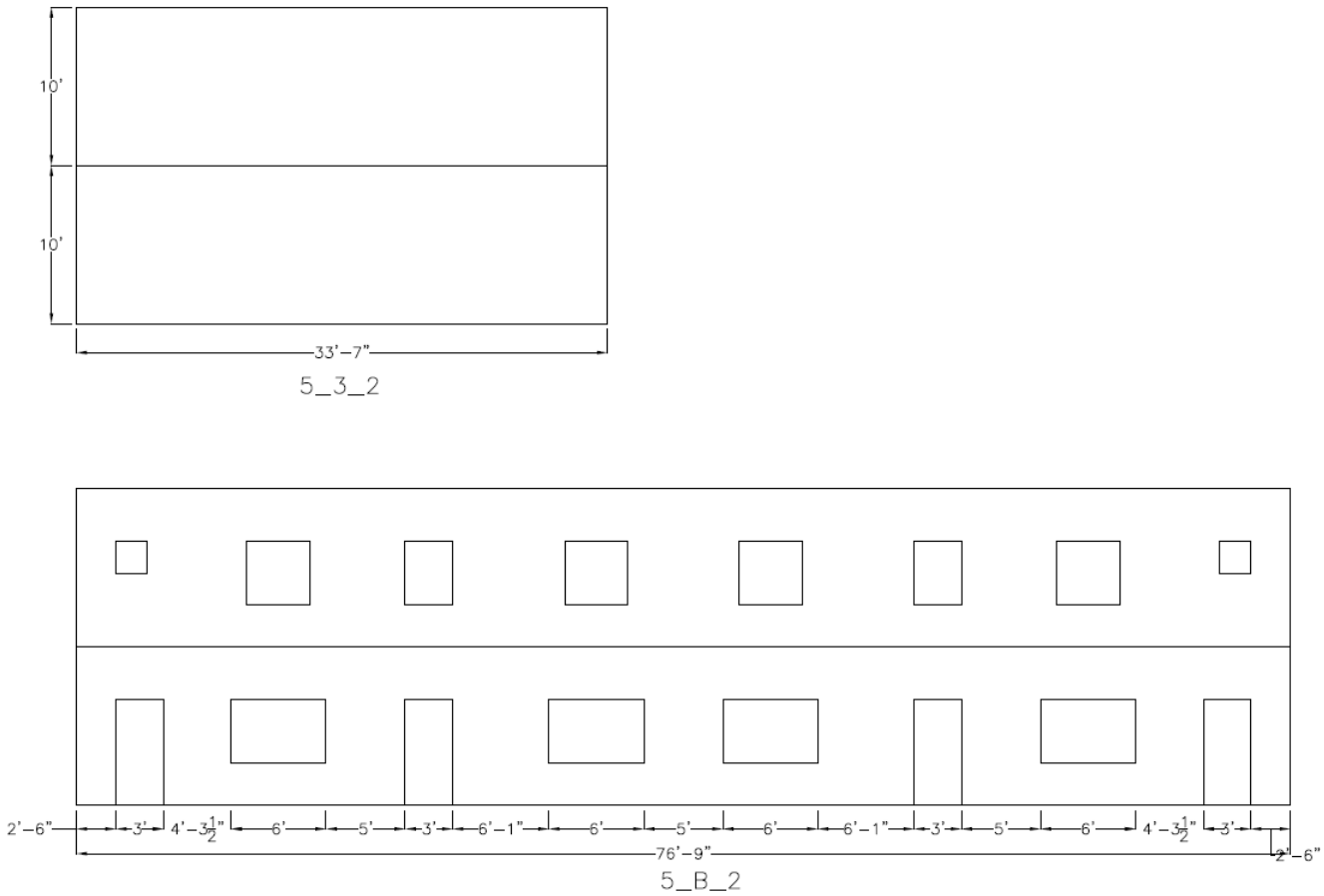


Figure B81. Index Building 5 extracted shear wall lines.

Table B69—Extracted archetypes, Index Building 5

Group No.	Basic Config.		Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-25	20ft-60ft wall	Low aspect ratio panels	High		5_B_2_2_ LR_HG_ DX_SP	75
PG-29			Low		5_B_2_2_ LR_LG_ DX_SP	87
PG-41		Mixed aspect ratio	High		5_B_2_2_ MR_HG_ DX_SP	123
PG-45			Low		5_B_2_2_ MR_LG_ DX_SP	135

Table B70—Seismic weight detailed calculation, Index Building 5, 2-story

								Low gravity				High gravity			
Level	Story	h (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			2608					76327.5			132105	76327.5			132105
	2	10		259	2352.0	124.3	1243		79311	32244			79311	32244	
1			2608					81456.5			190618	128693.9			244262
	1	10		259	2210.0	124.3	1243		74523	32244			87337	32244	
Ground			0												

Table B71—Seismic weight summary, Index Building 5, 2-story

Level	h (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
2		132.1	132.1
	10		
1		190.6	244.3
	10		
Ground			

INPUT DATA

Total Height	h_n=	20.0	ft				
Total Weight	W=	323	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.2648					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	4,548					

DESIGN SUMMARY

Total base shear

V = 107.57

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	132	2,642	0.581	62.5		
	10.00						62.5	
1		10.0	191	1,906	0.419	45.1		625
	10.00						107.6	
		0.0						1,701

Seismic base shear calculation, low gravity, Index Building 5, 2-story, R = 3

INPUT DATA

Total Height	h_n=	20.0	ft				
Total Weight	W=	376	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.2648					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	5,085					

DESIGN SUMMARY

Total base shear

V = 125.47

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	132	2,642	0.520	65.2		
	10.00						65.2	
1		10.0	244	2,443	0.480	60.3		652
	10.00						125.5	
		0.0						1,907

Seismic base shear calculation, high gravity, Index Building 5, 2-story, R = 3

Table B72—Tributary load calculation, Index Building 5, 2-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 2	A	1320.70	0.500	62.49		65.19	
	B	**1320.70**	**0.500**		**31.24**		**32.59**
Story 1	A	1320.70	0.500	107.57		125.47	
	B	**1320.70**	**0.500**		**53.78**		**62.73**

Table B73—Archetype 75 (5_B_2_2_ LR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	3	A3	4	D	2084.0	2	7/8	0.69	1.33	1646.3	-	-	-
1	2	S	5	6.875	5	A3	7	D	3647.0	2	1 1/2	0.74	2.35	3127.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	5	A3	4	D	2084.0	2	7/8	0.69	1.31	1613.1	41.68	32.59	1.28
1	2	S	5	6.875	5	A3	7	D	3647.0	2	1 1/2	0.74	2.36	3145.7	72.94	62.73	1.16

*Values shown are for unreinforced option and before applying C_d.

Table B74—Archetype 87 (5_B_2_2_ LR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	3	A3	3	D	1563.0	2	3/4	0.75	1.16	1562.2	-	-	-
1	2	S	5	6.875	5	A3	6	D	3126.0	2	1 3/8	0.73	2.05	2689.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	3	A3	3	D	1563.0	2	3/4	0.75	1.16	1562.2	31.26	31.24	1.00
1	2	S	5	6.875	5	A3	6	D	3126.0	2	1 3/8	0.73	2.05	2689.2	62.52	53.78	1.16

*Values shown are for unreinforced option and before applying C_d.

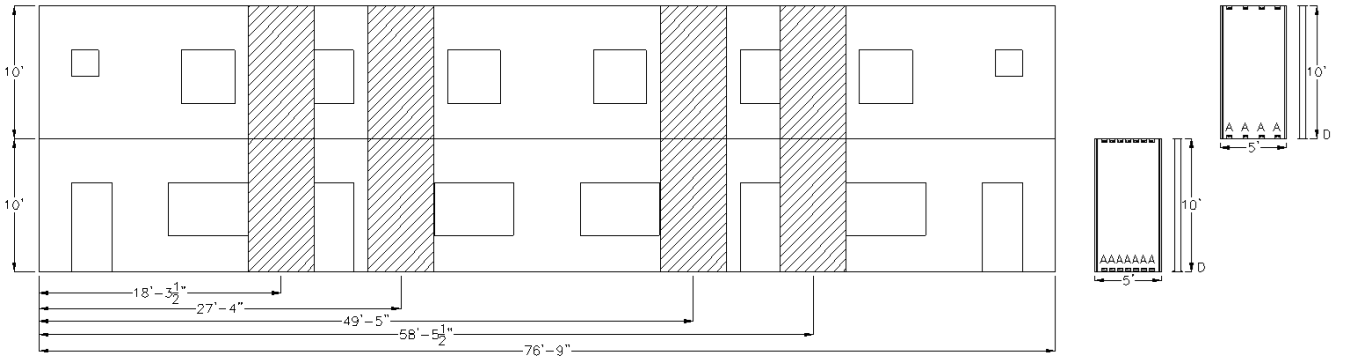


Figure B82. Archetype 75.

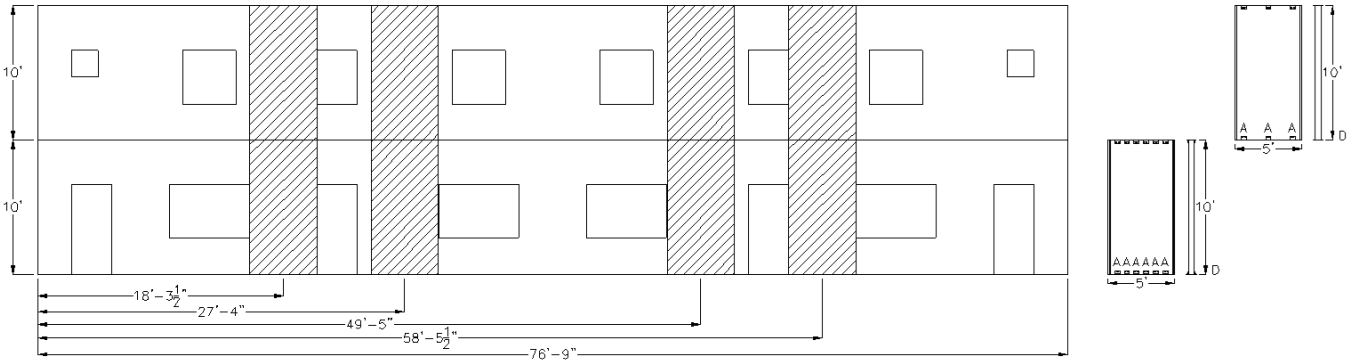


Figure B83. Archetype 87.

Table B75—Archetype 123 (5_B_2_2_ MR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.80	0.48	695.2	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/4	0.98	1.10	1940.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.80	0.48	695.2	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/4	0.98	1.10	1940.6	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	3	A3	5	D	2605.0	2	1	0.80	1.78	2564.2	36.47	32.59	1.12
1	2	S	5	6.875	5	A3	9	D	4689.0	2	1 3/8	0.98	2.47	4332.8	67.73	62.73	1.08

Table B76—Archetype 135 (5_B_2_2_ MR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.90	0.52	846.2	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/8	1.00	1.04	1884.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.90	0.52	846.2	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/8	1.00	1.04	1884.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	4.125	3	A3	5	D	2605.0	2	3/4	0.90	1.41	2278.2	36.47	31.24	1.17
1	2	S	5	6.875	5	A3	7	D	3647.0	2	1 1/8	1.00	1.94	3494.1	57.31	53.78	1.07

* Values show are for unreinforced option and before applying C_d.

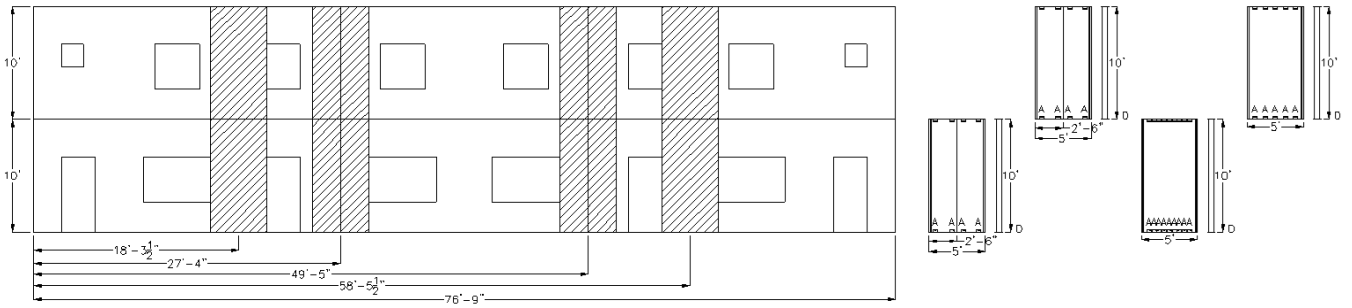


Figure B84. Archetype 123.

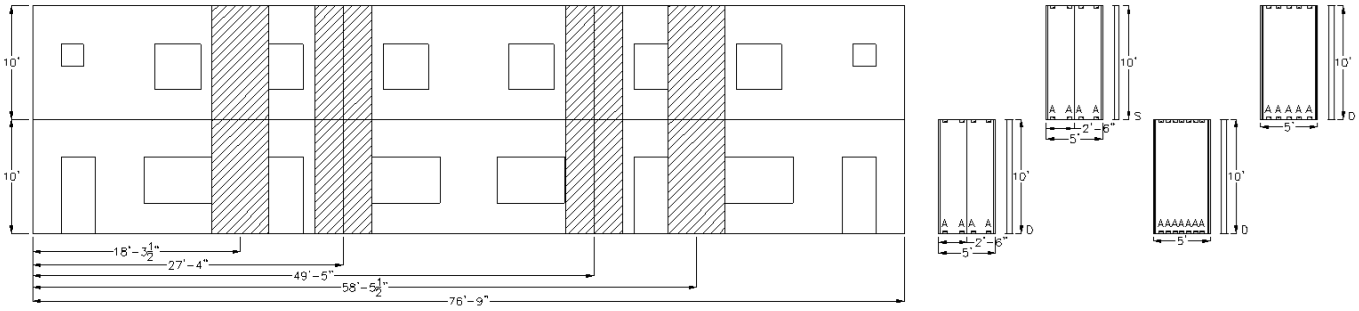


Figure B85. Archetype 135.

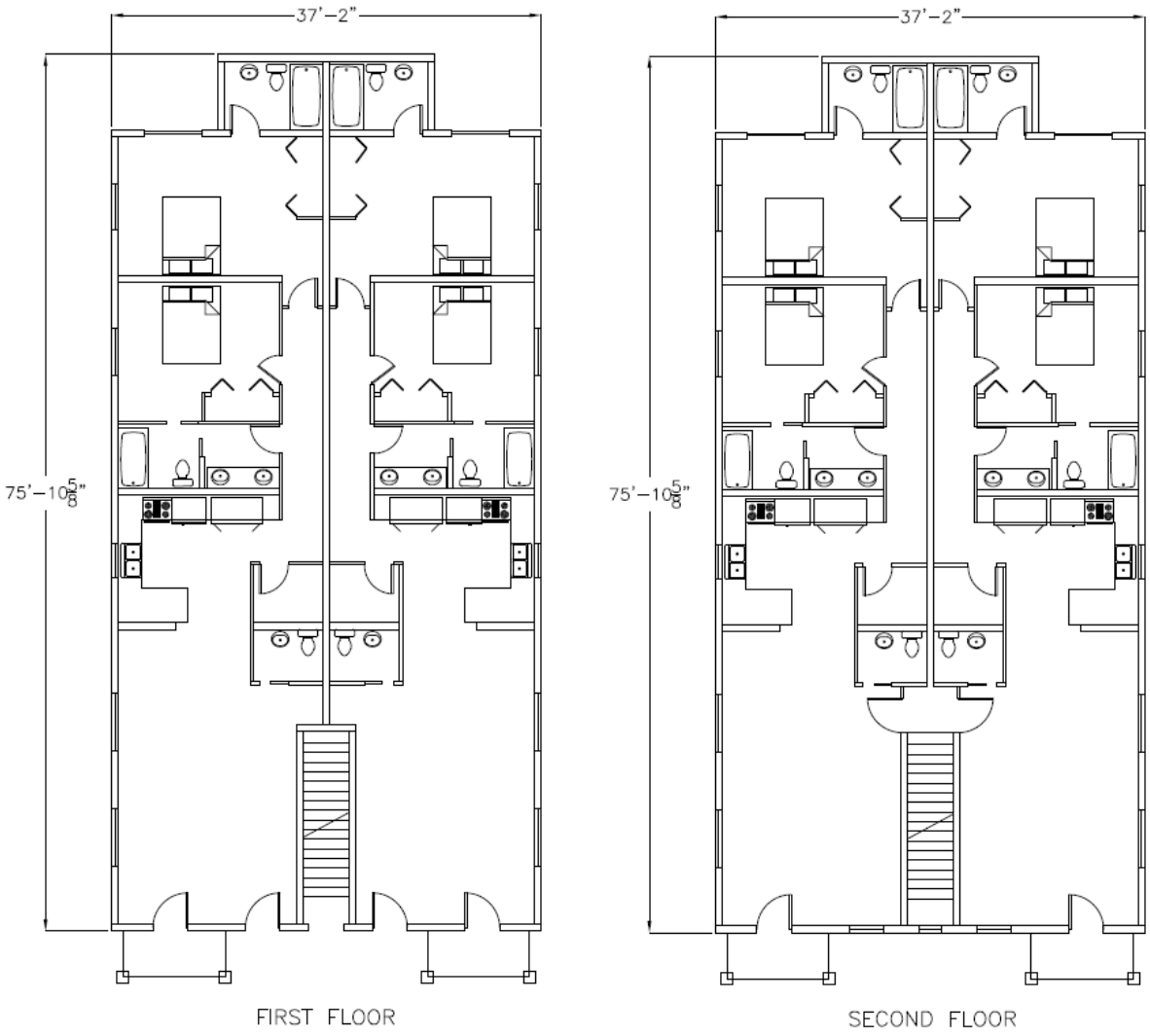


Figure B86. Index Building 6 floor plan.



Figure B87. Index Building 6 assigned shear walls.

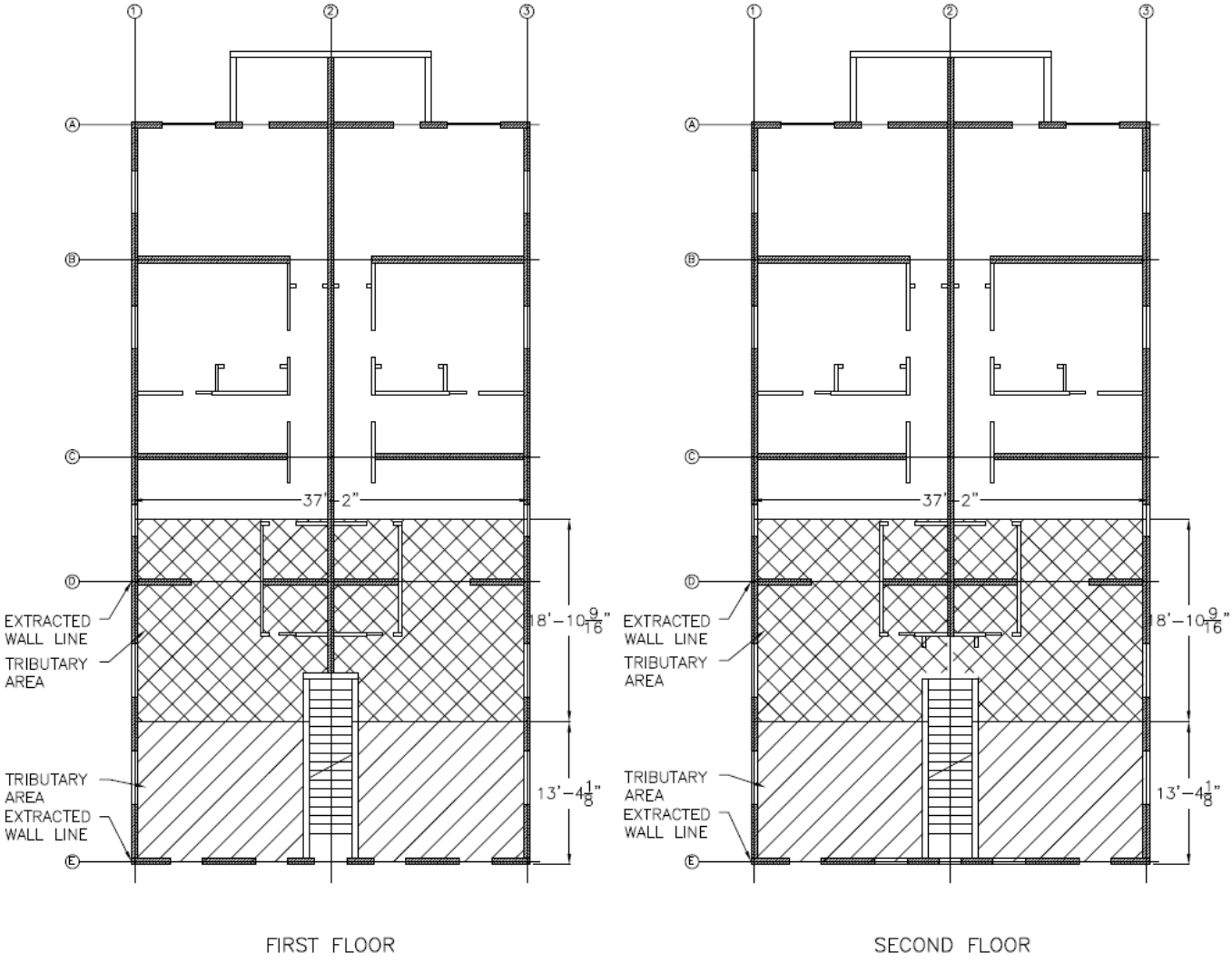


Figure B88. Index Building 6 tributary area.

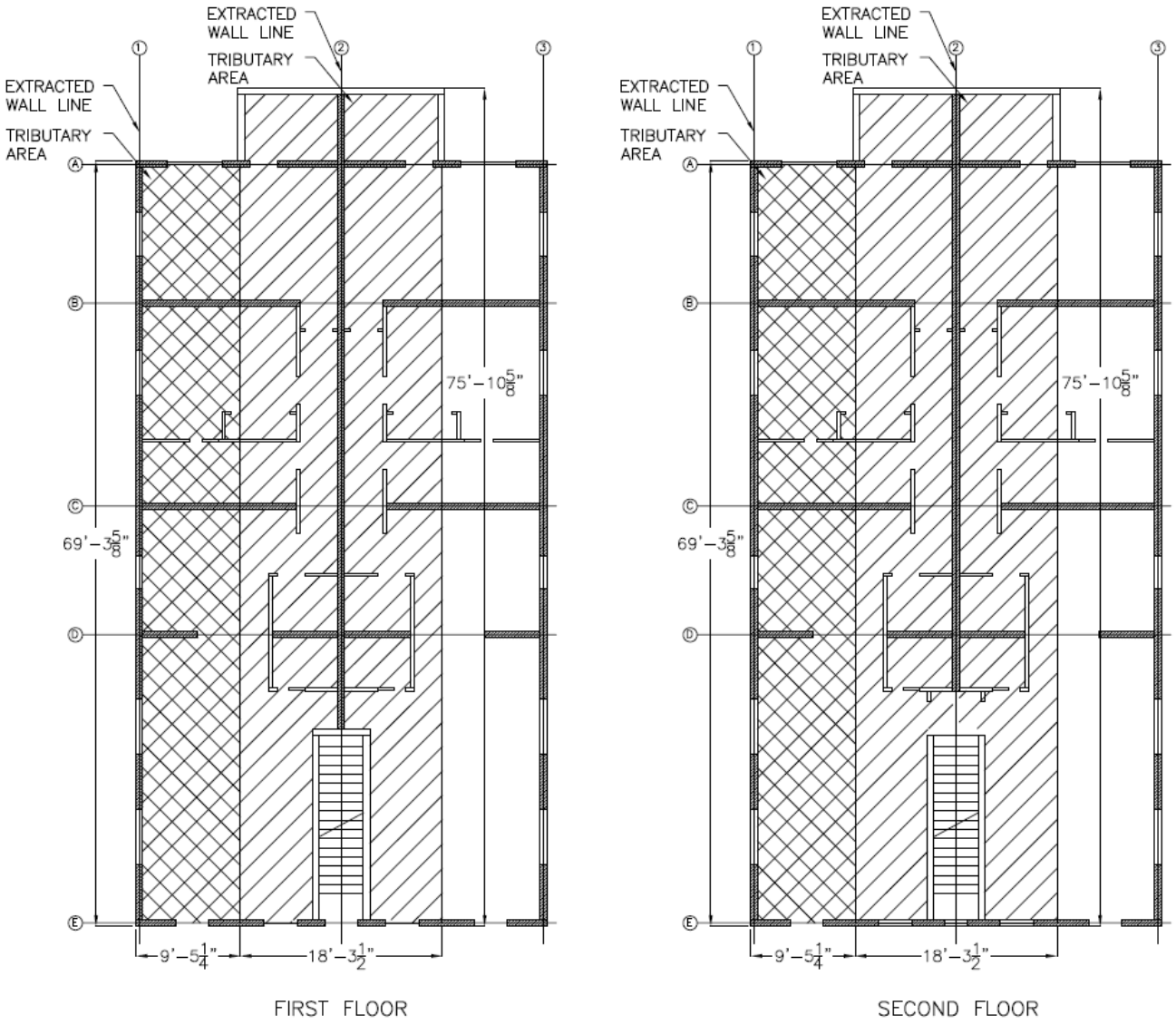


Figure B89. Index Building 6 tributary area.

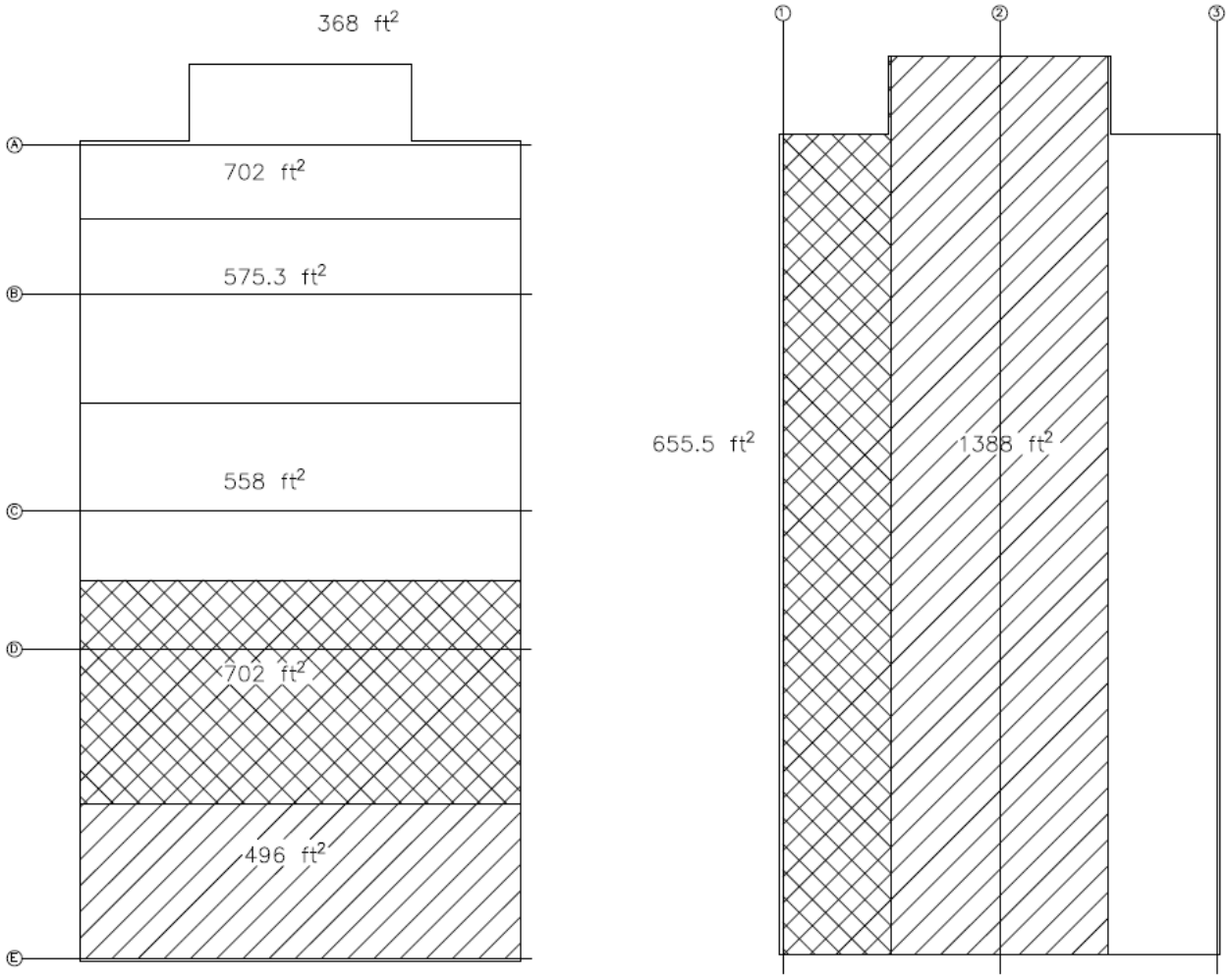


Figure B90. Index Building 6 tributary area.

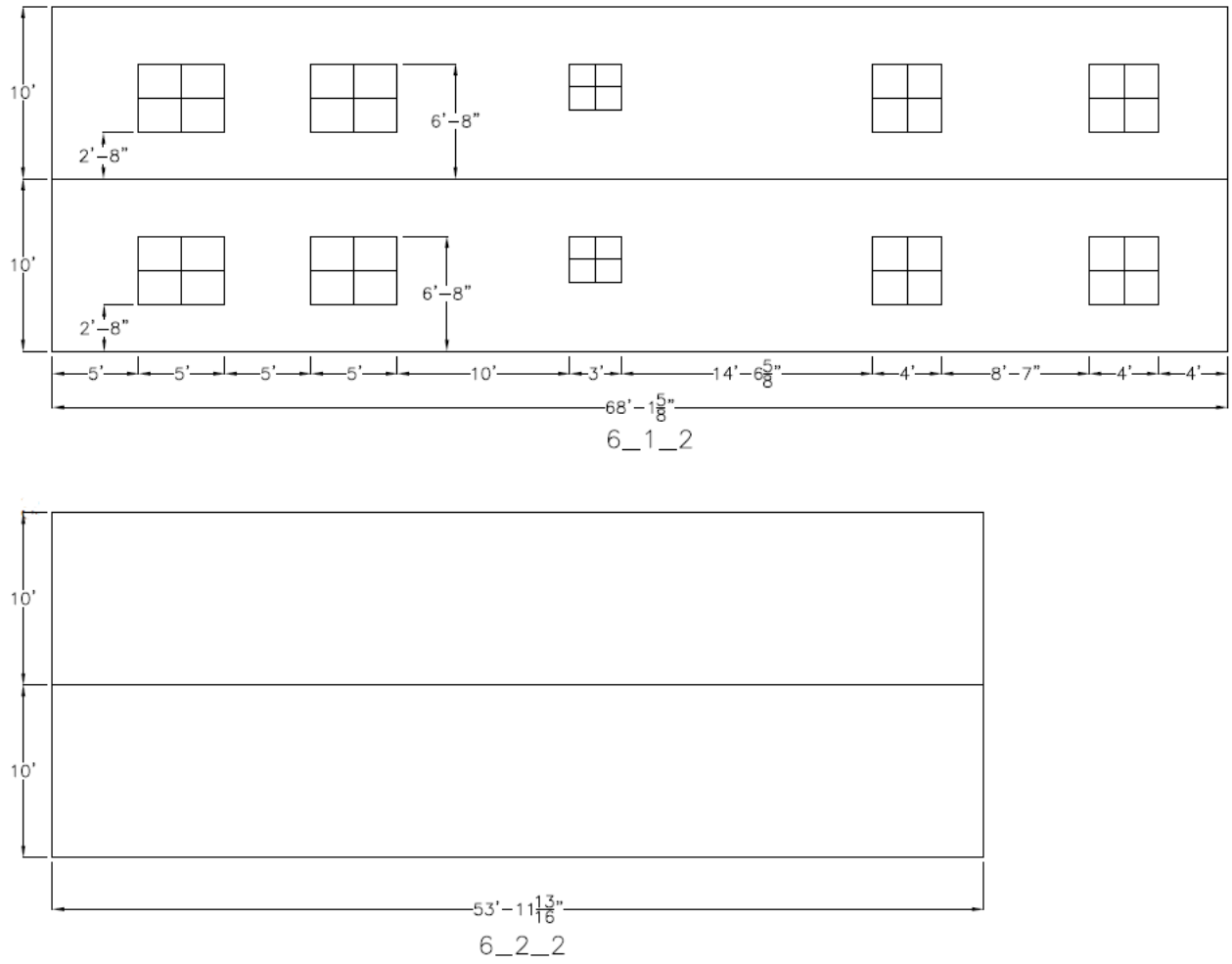


Figure B91. Index Building 6 extracted shear wall lines.

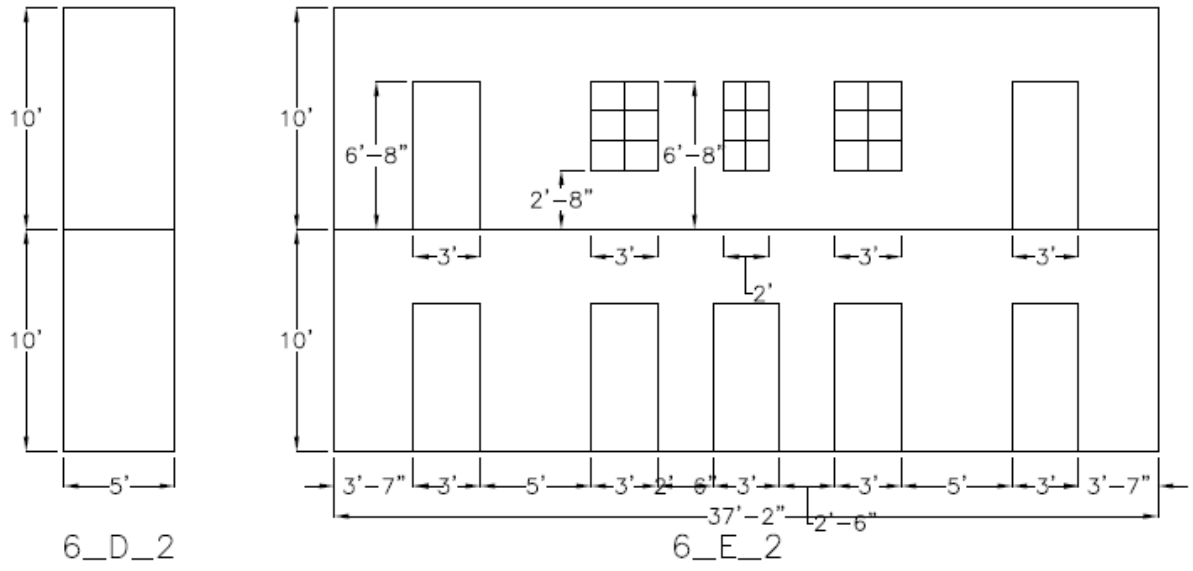


Figure B92. Index Building 6 extracted shear wall lines.

Table B77—Extracted archetypes, Index Building 6

Group	Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype
PG-1	2.5ft-20ft wall, Low aspect ratio panels	High	SDC D_{max}	6_D_2_1_ LR_HG_DX_SP	3
PG-5		Low		6_D_2_1_ LR_LG_DX_SP	15
PG-9	2.5ft-20ft wall, High aspect ratio panels	High		6_D_2_1_HR_HG_DX_SP	27
PG-13		Low		6_D_2_1_ HR_LG_DX_SP	39
PG-33	20ft-60ft wall, High aspect ratio panels	High		6_E_2_2_ HR_HG_ DX_SP	98
PG-37		Low		6_E_2_2_ HR_LG_ DX_SP	110

Table B78—Seismic weight detailed calculation, Index Building 6, 2-story

Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$ Openings	$L_{intwall}$	$A_{IntWall}$ Openings	Low gravity: W_{floor} (lb)	Low gravity: $W_{extWalls}$ (lb)	Low gravity: $W_{IntWalls}$ (lb)	Low gravity: ΣW_{Level} (lb)	High gravity: W_{floor} (lb)	High gravity: $W_{extWalls}$ (lb)	High gravity: $W_{IntWalls}$ (lb)	High gravity: ΣW_{Level} (lb)
Roof			2700					79020			134472	79020			134472
	2	10		226	1985.3	169.5	1695		66947	43957			66947	43957	
1			2700					84330			195234	133234			244138
	1	10		226	1985.3	169.50	1695		66947	43957			66947	43957	
Ground			0												

Table B79—Seismic weight summary, Index Building 6, 2-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
2		134.5	134.5
	10		
1		195.2	244.1
	10		
Ground			

INPUT DATA

Total Height	h_n=	20.0	ft					
Total Weight	W=	330	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	3						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.3333						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	4,642						

DESIGN SUMMARY

Total base shear

V = 109.90

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	135	2,690	0.579	63.7		
	10.00						63.7	
1		10.0	195	1,952	0.421	46.2		637
	10.00						109.9	
		0.0						1,736

Seismic base shear calculation, low gravity, Index Building 6, 2-story, R = 3

INPUT DATA

Total Height	h_n=	20.0	ft					
Total Weight	W=	379	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms}	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1}	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	3						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.3333						
	k =	1.00	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	5,131						

DESIGN SUMMARY

Total base shear

V = 126.20

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	135	2,690	0.524	66.2		
	10.00						66.2	
1		10.0	244	2,441	0.476	60.0		662
	10.00						126.2	
		0.0						1,924

Seismic base shear calculation, high gravity, Index Building 6, 2-story, R = 3

Table B80—Tributary load calculation, Index Building 6, 2-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 2	A	369	0.136	63.69		66.00	
	B	575	0.213				
	C	558	0.207				
	D	**702**	**0.260**		**16.56**		**17.16**
	E	**496**	**0.184**		**11.70**		**12.13**
Story 1	A	369	0.136	109.90		126.00	
	B	575	0.213				
	C	558	0.207				
	D	**702**	**0.260**		**28.58**		**32.76**
	E	**496**	**0.184**		**20.19**		**23.15**

Table B81—Archetype 3 (6_D_2_1_ LR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.64	0.74	858.1	5.21	4.29	1.21
1	1	S	5	4.125	3	A3	4	D	2084.0	2	1 1/8	0.70	1.31	1638.1	10.42	8.19	1.27

Table B82—Archetype 15 (6_D_2_1_ LR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	4.125	3	A3	4	S	1042.0	2	5/8	0.63	0.73	828.0	5.21	4.14	1.26
1	1	S	5	4.125	3	A3	6	S	1563.0	2	1	0.71	1.12	1428.8	7.82	7.14	1.09

*Values shown are for unreinforced option and before applying C_d.

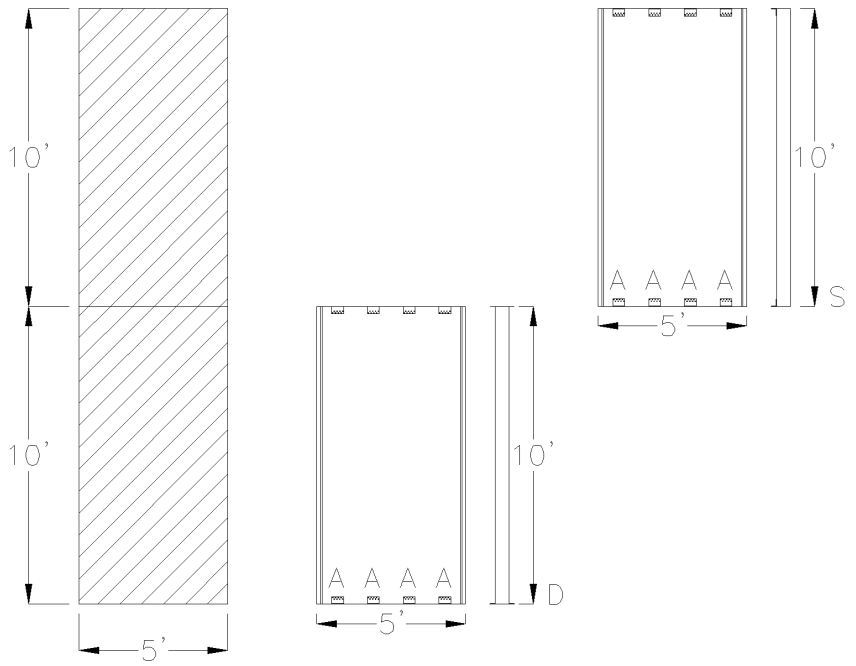


Figure B93. Archetype 3.

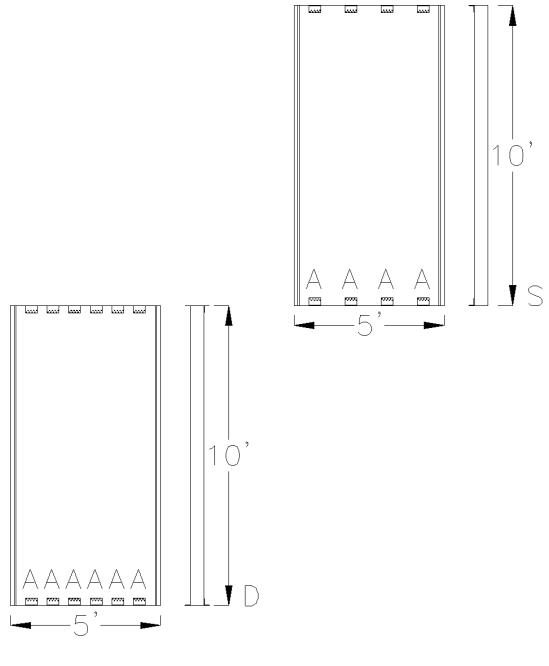


Figure B94. Archetype 15.

Table B83—Archetype 27 (6_D_2_1_HR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.91	0.53	858.1	5.21	4.29	1.21
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/8	0.93	0.98	1638.1	10.42	8.19	1.27

Table B84—Archetype 39 (6_D_2_1_ HR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.89	0.52	828.0	5.21	4.14	1.26
1	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/8	0.96	0.82	1428.8	10.42	7.14	1.46

*Values shown are for unreinforced option and before applying C_d.

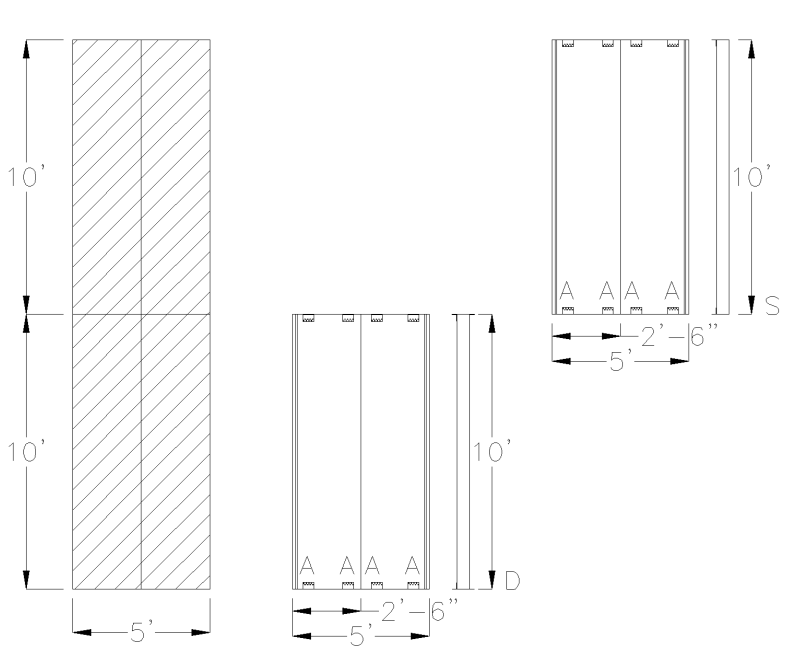


Figure B95. Archetype 27.

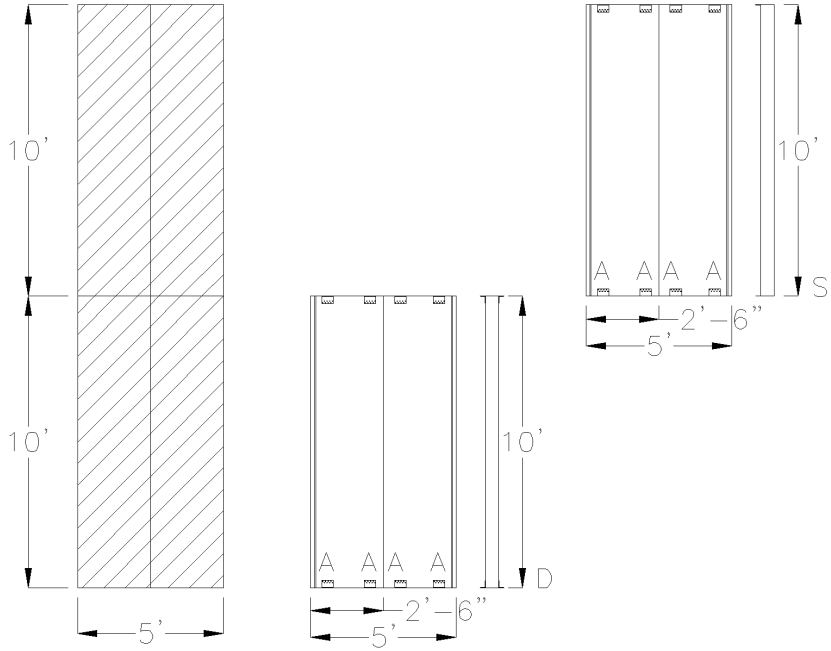


Figure B96. Archetype 39.

Table B85—Archetype 98 (6_E_2_2_ HR_HG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.83	0.81	1212.6	-	-	-
1	2	M	2.5	6.875	5	A3	3	D	3126.0	2	1 1/2	0.97	1.32	2314.7	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.83	0.81	1212.4	20.84	12.13	1.72
1	2	M	2.5	6.875	5	A3	3	D	3126.0	2	1 1/2	0.97	1.32	2315.0	31.26	23.15	1.35

*Values shown are for unreinforced option and before applying C_d.

Table B86—Archetype 110 (6_E_2_2_ HR_LG_ DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.81	0.80	1170.0	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.98	1.14	2019.0	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	D	2084.0	2	1	0.81	0.80	1170.0	20.84	11.70	1.78
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.98	1.14	2019.1	20.84	20.19	1.03

*Values shown are for unreinforced option and before applying C_d.

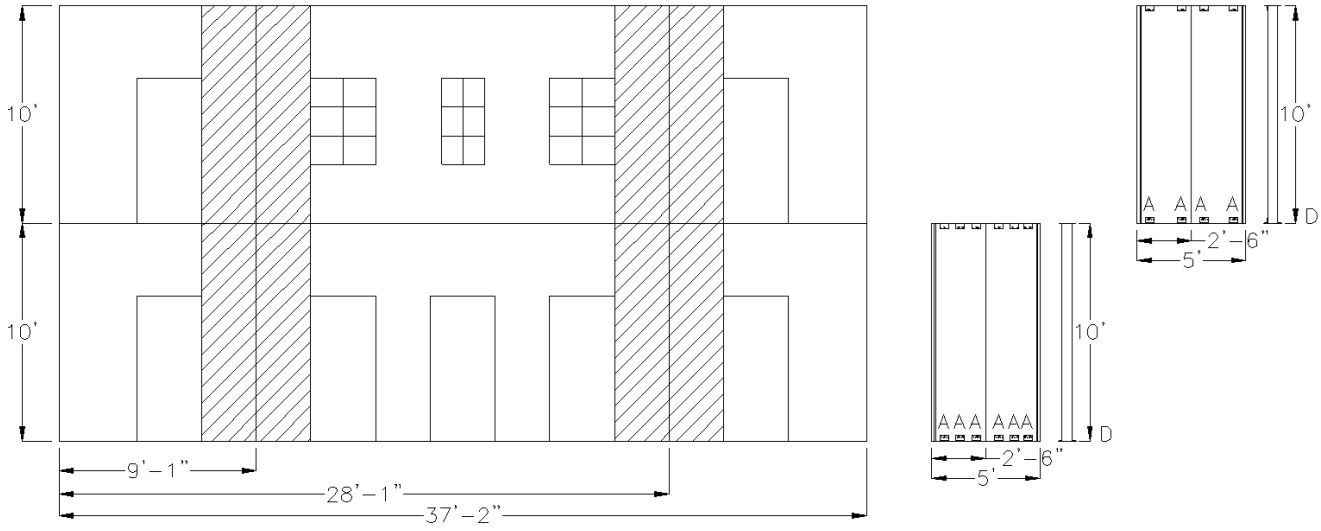


Figure B97. Archetype 98.

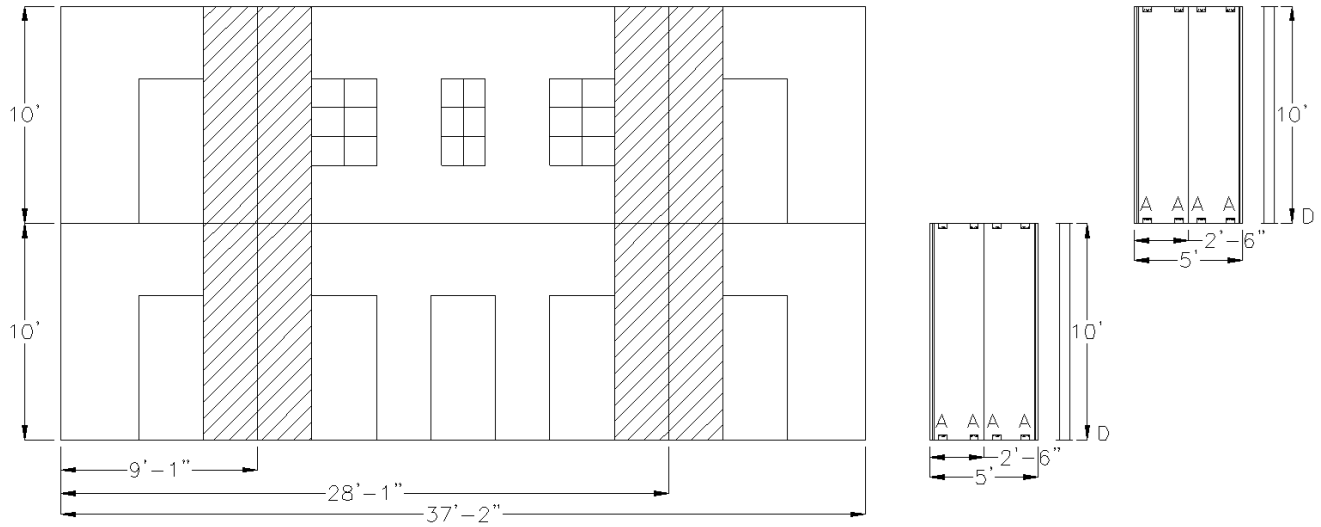


Figure B98. Archetype 110.

INPUT DATA

Total Height	h_n=	**20.0**	ft					
Total Weight	W=	**379**	k					
Seismic Design Category		**Dmax**						
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)					
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**		
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**		
	S_{DS} =	**1.000**	g ,					
	S_{D1} =	**0.600**	g					
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**						
The coefficient(ASCE Tab. 12.2.1)	R =	**4**						
	x =	**0.75**	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	**1.40**						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.25						
	k =	**1.00**	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	5,131						

DESIGN SUMMARY

Total base shear

V = 94.65

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	**135**	2,690	0.524	49.6		
	10.00						49.6	
1		10.0	**244**	2,441	0.476	45.0		496
	10.00						94.7	
		0.0						1,443

Seismic base shear calculation, low gravity, Index Building 6, 2-story, R = 4

INPUT DATA

Total Height	h_n=	**20.0**	ft					
Total Weight	W=	**330**	k					
Seismic Design Category		**Dmax**						
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)					
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**		
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**		
	S_{DS} =	**1.000**	g ,					
	S_{D1} =	**0.600**	g					
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**						
The coefficient(ASCE Tab. 12.2.1)	R =	**4**						
	x =	**0.75**	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.19	Sec, (ASCE 12.8.2.1)					
	Cu=	**1.40**						
	T=Cu*Ta=	0.2648						
	Ts=	0.6						
	Cs=	0.25						
	k =	**1.00**	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	4,642						

DESIGN SUMMARY

Total base shear

V = 82.43

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
2		20.0	**135**	2,690	0.579	47.8		
	10.00						47.8	
1		10.0	**195**	1,952	0.421	34.7		478
	10.00						82.4	
		0.0						1,302

Seismic base shear calculation, high gravity, Index Building 6, 2-story, R = 4

Table B87—Tributary load calculation, Index Building 6, 2-story, E-W direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 2	A	369	0.136	47.76		49.62	
	B	575	0.213				
	C	558	0.207				
	D	**702**	**0.260**		**12.42**		**12.90**
	E	**496**	**0.184**		**8.78**		**9.12**
Story 1	A	369	0.136	82.43		94.65	
	B	575	0.213				
	C	558	0.207				
	D	**702**	**0.260**		**21.43**		**24.61**
	E	**496**	**0.184**		**15.14**		**17.39**

Table B88—Archetype 27 (6_D_2_1_HR_HG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.77	0.46	645.1	5.21	3.23	1.62
1	2	M	2.5	4.125	3	A3	3	S	1563.0	2	1	0.95	0.72	1230.5	7.82	6.15	1.27

Table B89—Archetype 39 (6_D_2_1_ HR_LG_DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.76	0.46	621.0	5.21	3.10	1.68
1	2	M	2.5	4.125	3	A3	3	S	1563.0	2	1	0.88	0.68	1071.6	7.82	5.36	1.46

*Values shown are for unreinforced option and before applying C_d.

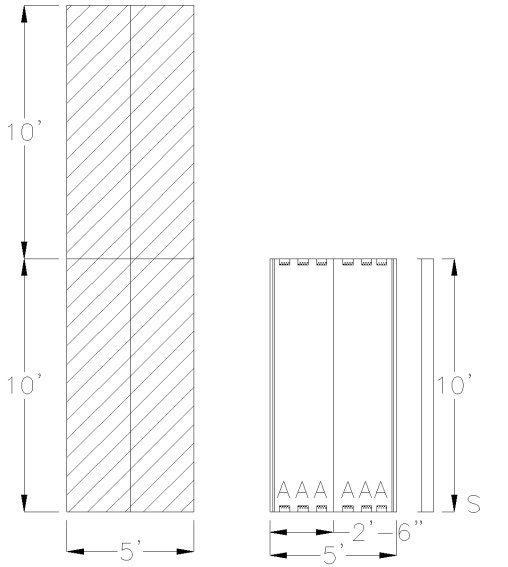


Figure B99. Archetype 27.

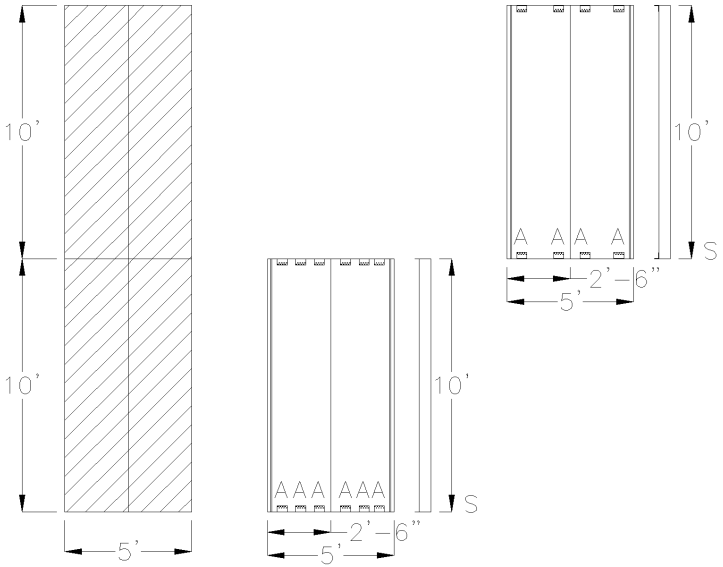


Figure B100. Archetype 39.

Table B90—Archetype 98 (6_E_2_2_ HR_HG_ DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.94	0.54	911.6	-	-	-
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/8	0.96	1.00	1738.9	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.94	0.54	911.6	10.42	9.12	1.14
1	2	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/8	0.96	1.00	1738.9	20.84	17.39	1.20

*Values shown are for unreinforced option and before applying C_d.

Table B91—Archetype 110 (6_E_2_2_ HR_LG_ DX_SP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.92	0.53	877.5	-	-	-
1	2	M	2.5	6.875	5	A3	3	S	1563.0	2	1	0.99	0.85	1514.3	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.92	0.53	877.5	10.42	8.78	1.19
1	2	M	2.5	6.875	5	A3	3	S	1563.0	2	1	0.99	0.85	1514.3	15.63	15.14	1.03

*Values shown are for unreinforced option and before applying C_d.

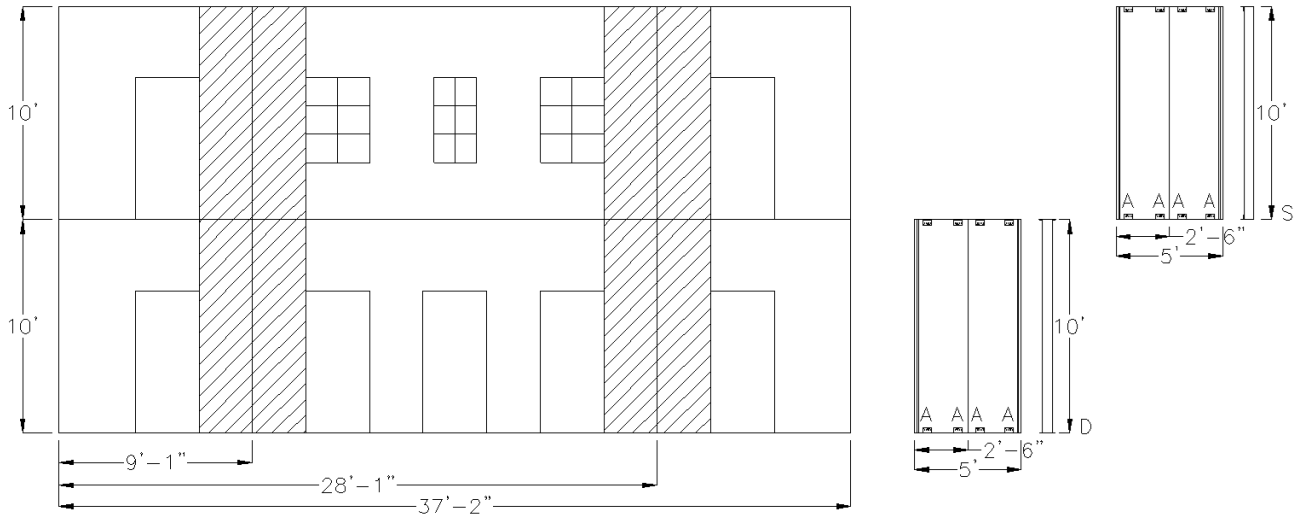


Figure B101. Archetype 98.

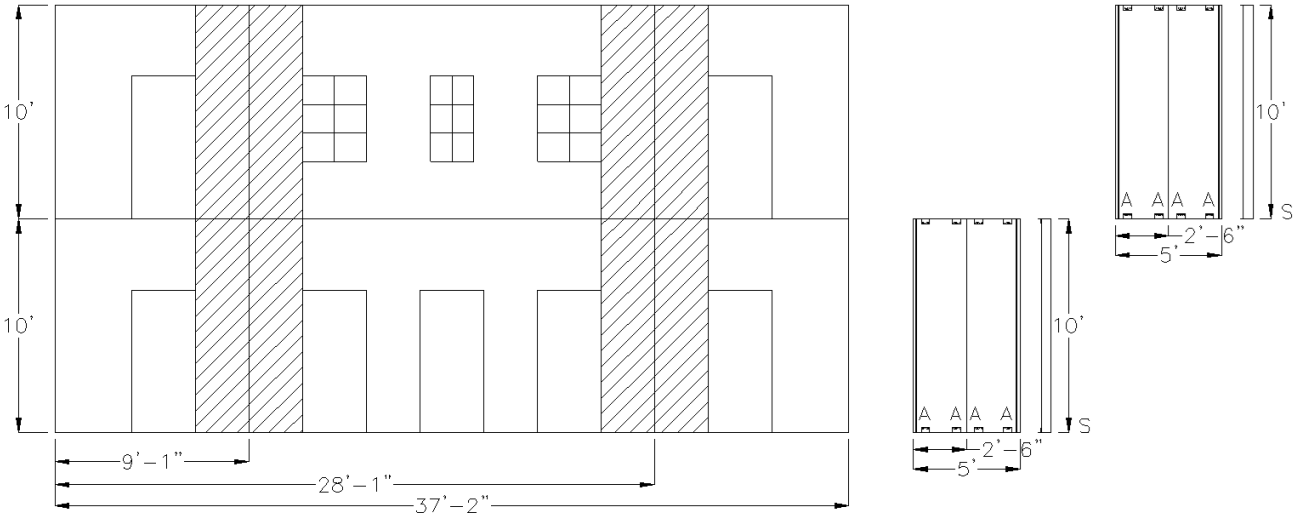


Figure B102. Archetype 110.

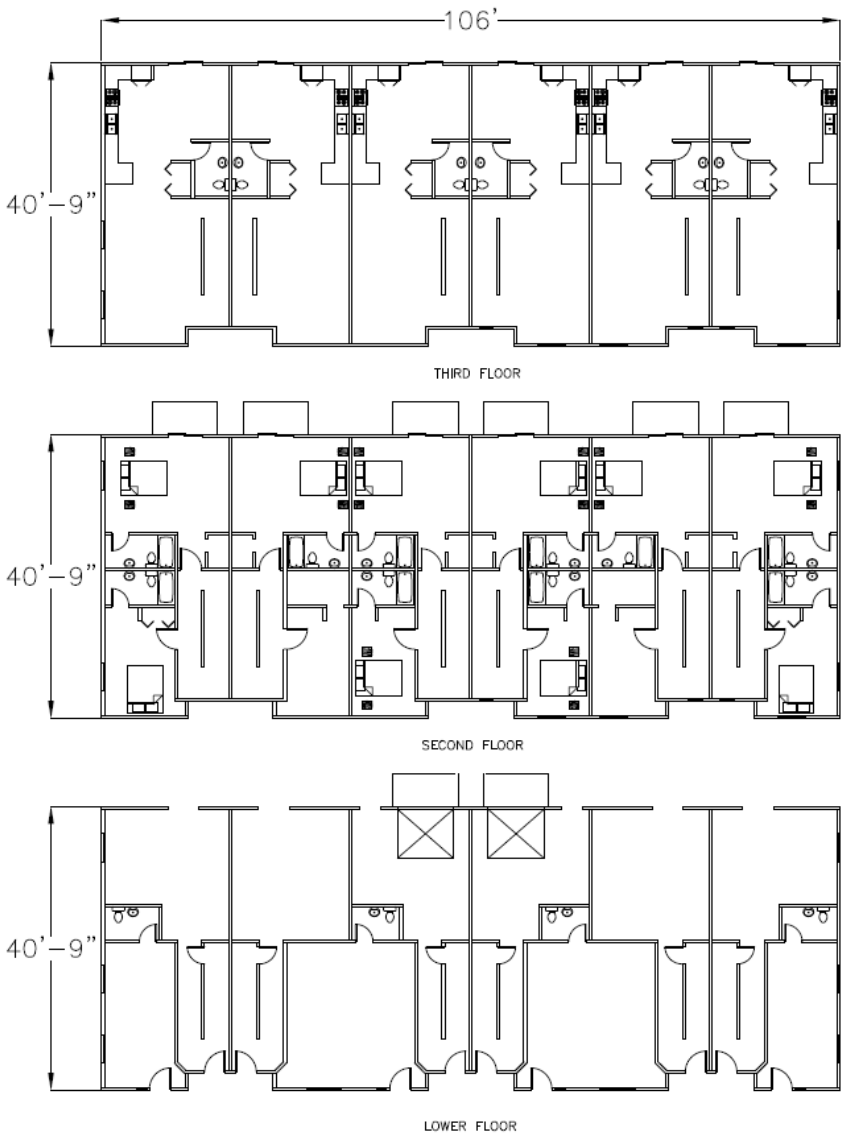


Figure B103. Index Building 7 floor plan.

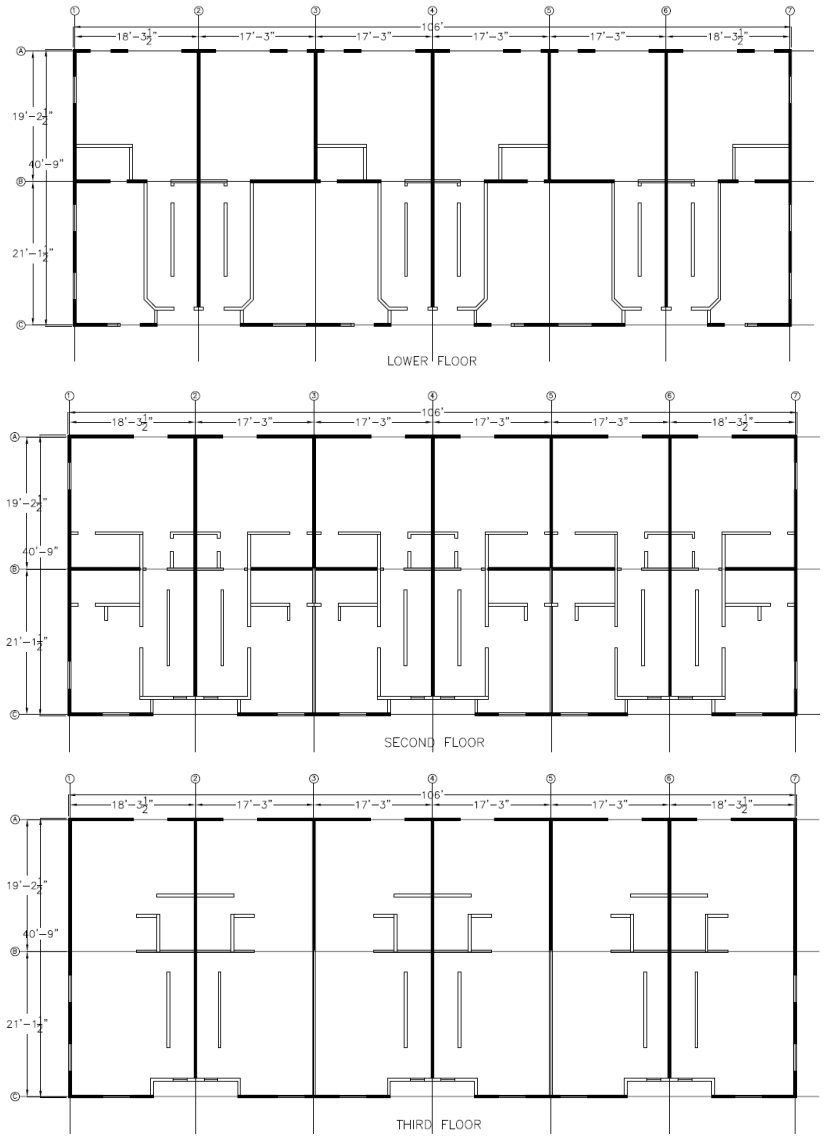


Figure B104. Index Building 7 assigned shear walls.

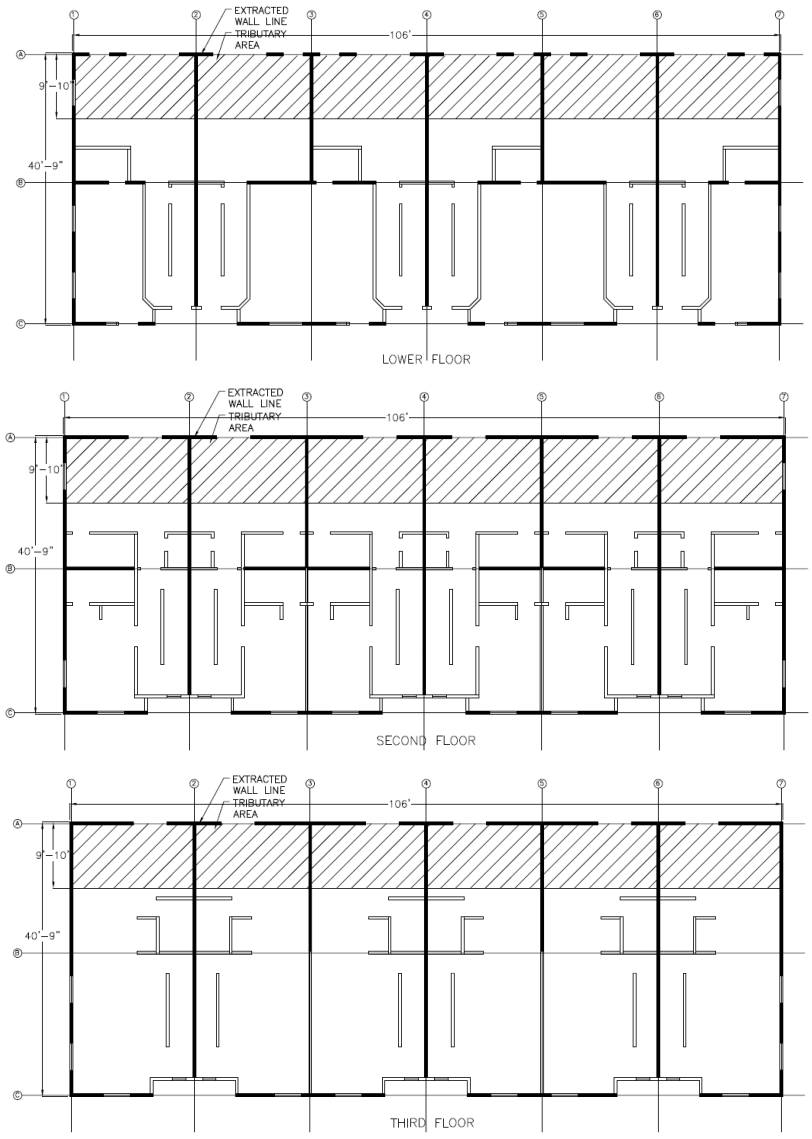


Figure B105. Index Building 7 tributary area.

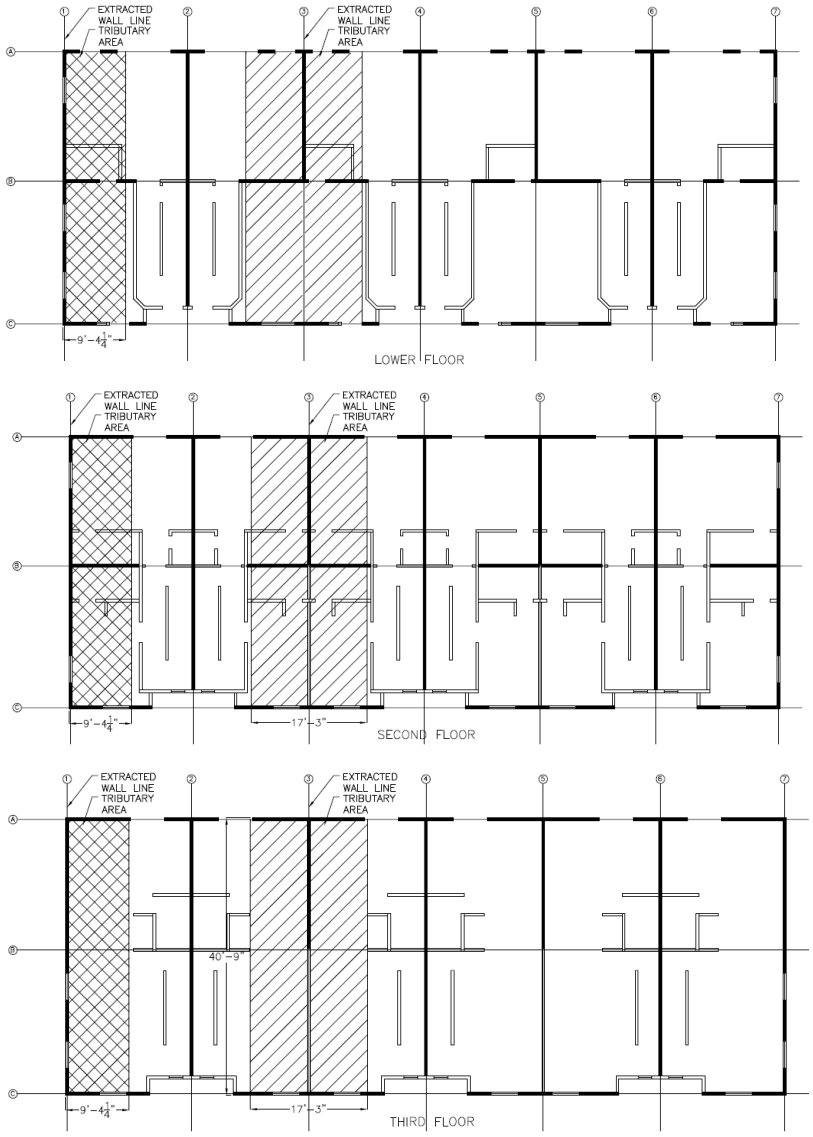


Figure B106. Index Building 7 tributary area.

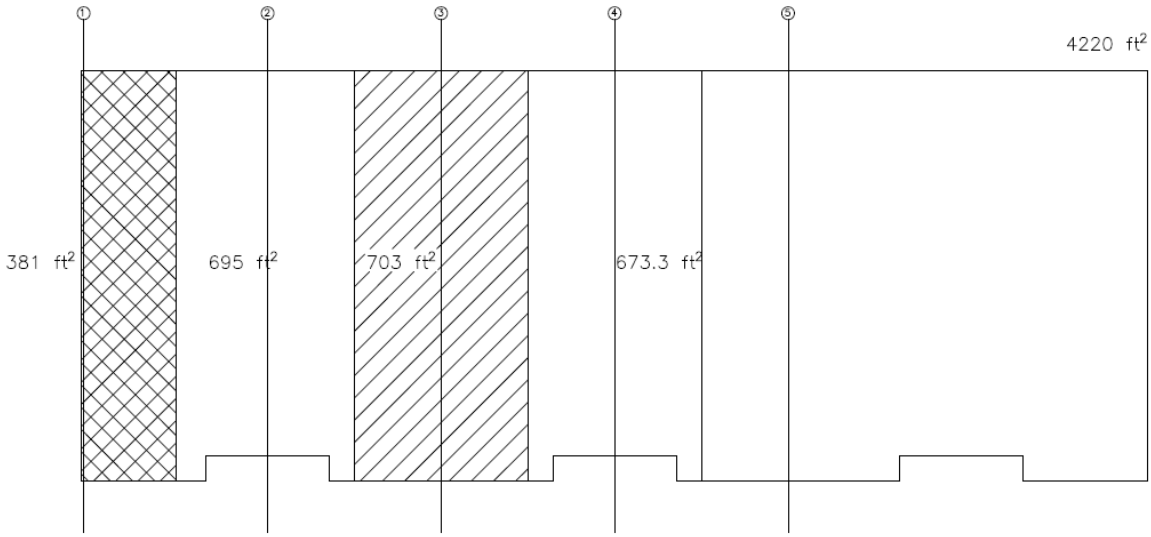


Figure B107. Index Building 7 tributary area.

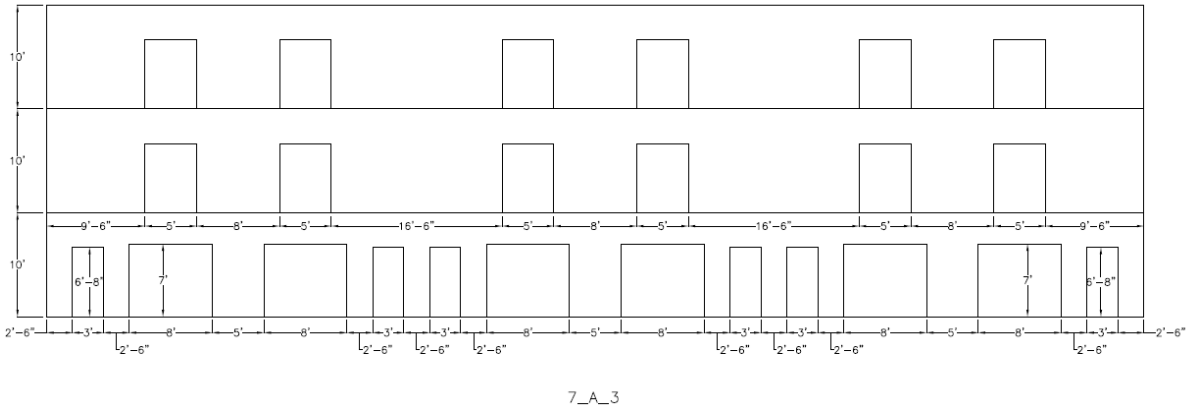


Figure B108. Index Building 7 extracted shear wall lines.

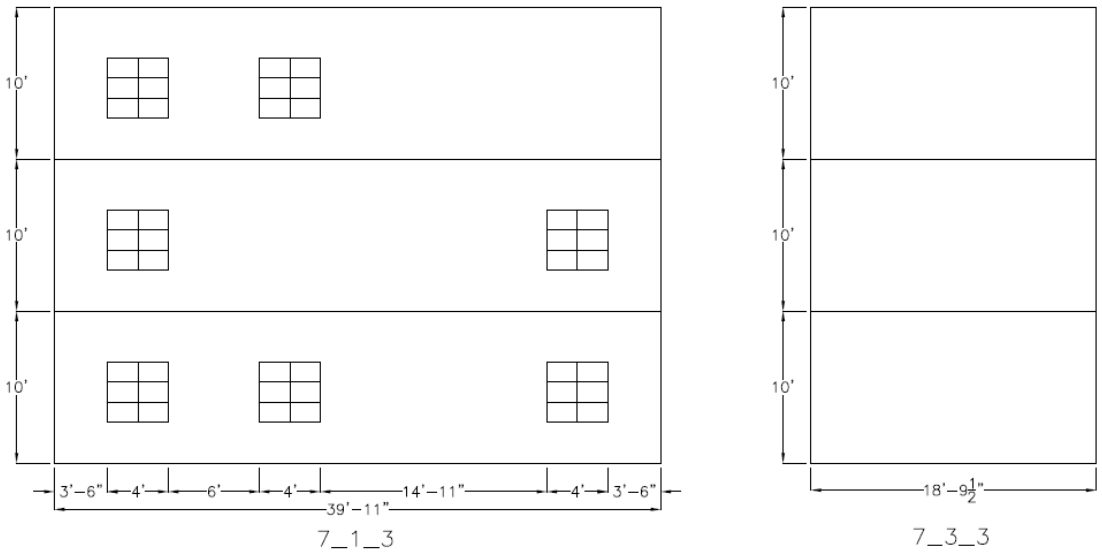


Figure B109. Index Building 7 extracted shear wall lines.

Table B92—Extracted archetypes, Index Building 7

Group No.	Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-10	2.5ft-20ft wall panels	High		7_3_3_1_MR_HG_DX_SP	51
PG-14	High aspect ratio	Low		7_3_3_1_MR_LG_DX_SP	63
PG-26	20ft-60ft wall panels	High		7_A_3_2_ HR_HG_ DX_SP	99
PG-30	Low aspect ratio	Low		7_A_3_2_ HR_LG_ DX_SP	111

Table B93—Seismic weight detailed calculation, Index Building 7, 3-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			4220					123505.3			207819	123505.3			207819
	3	10		304	2632	308	3080		88753	79875			88753	79875	
2			4220					131804.7			307759	208239.4			384193
	2	10		304	2632	402.5	3645		88753	94527			88753	94527	
1			4220					131804.7			306292	208239.4			382727
	1	10		304	2231	369.5	3488		75231	90464			75231	90464	
Ground			0												

Table B94—Seismic weight summary, Index Building 7, 3-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		207.8	207.8
	10		
2		307.8	384.2
	10		
1		306.3	382.7
	10		
Ground			

INPUT DATA

Total Height	h_n=	30.0	ft				
Total Weight	W=	822	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.3589					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	15,453					

DESIGN SUMMARY

Total base shear

V = 273.97

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	208	6,234	0.403	110.5		
	10.00						110.5	
2		20.0	308	6,156	0.398	109.1		1,105
	10.00						219.7	
1		10.0	306	3,063	0.198	54.3		3,302
	10.00						274.0	
		0.0						6,042

Seismic base shear calculation, low gravity, Index Building 7, 3-story, R = 3

INPUT DATA

Total Height	h_n=	30.0	ft				
Total Weight	W=	975	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.3589					
	Ts=	0.6					
	Cs=	0.3333					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	17,745					

DESIGN SUMMARY

Total base shear

V = 324.90

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	208	6,234	0.351	114.1		
	10.00						114.1	
2		20.0	384	7,684	0.433	140.7		1,141
	10.00						254.8	
1		10.0	383	3,827	0.216	70.1		3,690
	10.00						324.9	
		0.0						6,939

Seismic base shear calculation, high gravity, Index Building 7, 3-story, R = 3

Table B95—Tributary load calculation, Index Building 7, 3-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 3	1	381.0	0.090	110.52		114.14	
	2	695.0	0.164				
	3	**703.0**	**0.166**		**18.4**		**19.0**
	4	673.3	0.159				
	5	703.0	0.166				
	6	695.0	0.164				
	7	381.0	0.090				
Story 2	1	381.0	0.090	219.66		254.83	
	2	695.0	0.164				
	3	**703.0**	**0.166**		**36.5**		**42.3**
	4	673.3	0.159				
	5	703.0	0.166				
	6	695.0	0.164				
	7	381.0	0.090				
Story 1	1	381.0	0.090	273.97		324.90	
	2	695.0	0.164				
	3	**703.0**	**0.166**		**45.5**		**54.0**
	4	673.3	0.159				
	5	703.0	0.166				
	6	695.0	0.164				
	7	381.0	0.090				

Table B96—Tributary load calculation, Index Building 7, 3-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 3	**A**	**1042.00**	**0.244**	110.52	**26.95**	114.14	**27.84**
	B	2180.00	0.510				
	C	1050.70	0.246				
Story 2	**A**	**1042.00**	**0.244**	219.66	**53.57**	254.83	**62.15**
	B	2180.00	0.510				
	C	1050.70	0.246				
Story 1	**A**	**1042.00**	**0.244**	273.97	**66.81**	324.90	**79.23**
	B	2180.00	0.510				
	C	1050.70	0.246				

Table B97—Archetype 51 (7_3_3_1_MR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	4.125	3	A3	3	D	1563.0	4	5/8	0.65	1.17	1369.8	-	-	-
2	1	S	5	6.875	5	A3	8	D	4168.0	4	1 1/8	0.74	2.61	3463.0	-	-	-
1	1	S	5	9.625	7	A3	9	D	4689.0	4	1 1/2	0.80	3.18	4559.2	-	-	-
Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	4	M	2.5	4.125	3	A3	3	S	1563.0	2	7/8	0.65	2.07	1211.5	23.45	18.96	1.24
2	4	M	2.5	6.875	5	A3	3	D	3126.0	2	1 1/2	0.74	3.78	2502.3	52.10	42.34	1.23
1	4	M	2.5	9.625	7	A3	3	D	3126.0	2	1 3/4	0.80	4.35	3118.4	54.71	53.98	1.01

*Values shown are for unreinforced option and before applying C_d.

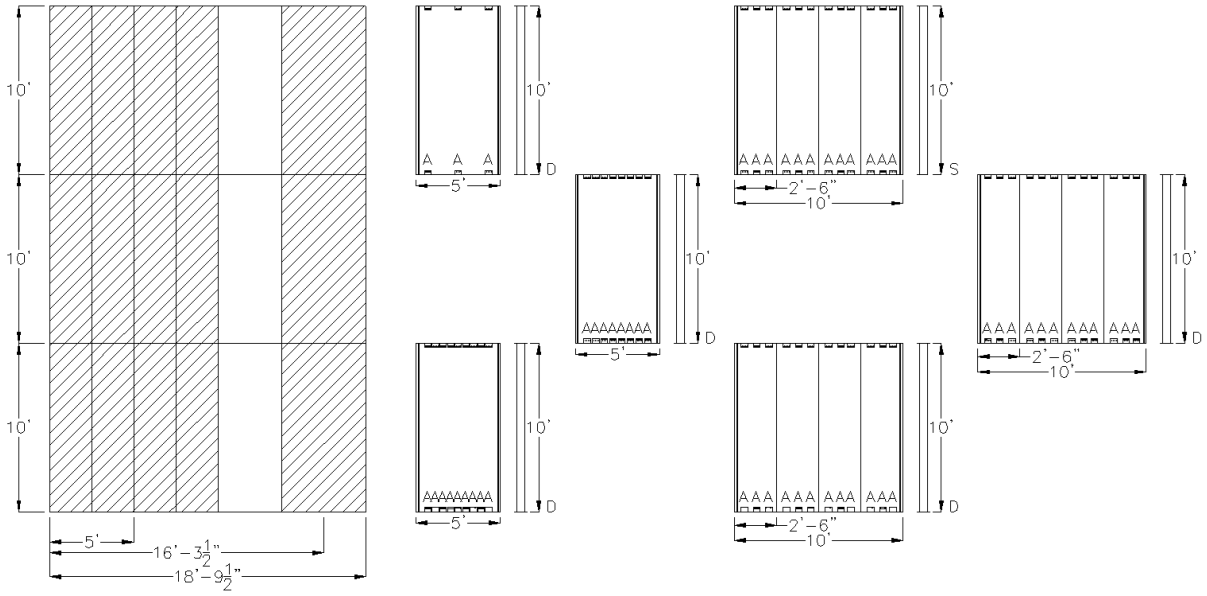


Figure B110. Archetype 51.

Table B98—Archetype 63 (7_3_3_1_MR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	4.125	3	A3	3	D	1563.0	2	5/8	0.42	1.98	1481.0	-	-	-
2	1	S	5	4.125	3	A3	6	D	3126.0	2	1	0.58	2.93	3085.7	-	-	-
1	1	S	5	6.875	5	A3	8	D	4168.0	2	1 3/8	0.54	4.12	4022.3	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	4	M	2.5	4.125	3	A3	3	S	1563.0	2	7/8	0.42	2.93	1095.7	23.45	18.36	1.28
2	4	M	2.5	4.125	3	A3	3	D	3126.0	2	1 3/8	0.58	4.00	2106.7	46.89	36.50	1.28
1	4	M	2.5	6.875	5	A3	3	D	3126.0	2	2	0.54	5.21	2540.6	52.10	45.52	1.14

*Values shown are for reinforced option and before applying C_d.

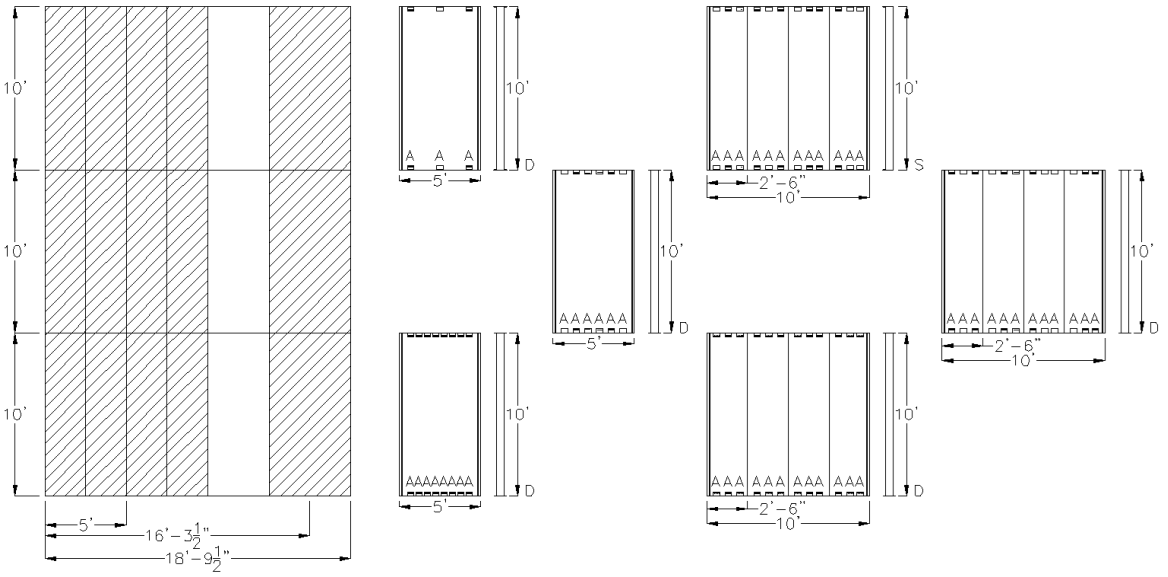


Figure B111. Archetype 63.

Table B99—Archetype 99 (7_A_3_2_ HR_HG_ DX_SP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.76	1.02	927.9	7.82	6.96	1.12
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/4	0.97	1.78	2071.5	15.63	15.54	1.01
1	3	M	2.5	6.875	5	A3	3	D	3126.0	2	1 3/4	0.88	2.49	2641.2	23.45	19.81	1.18

* There are four of (3)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

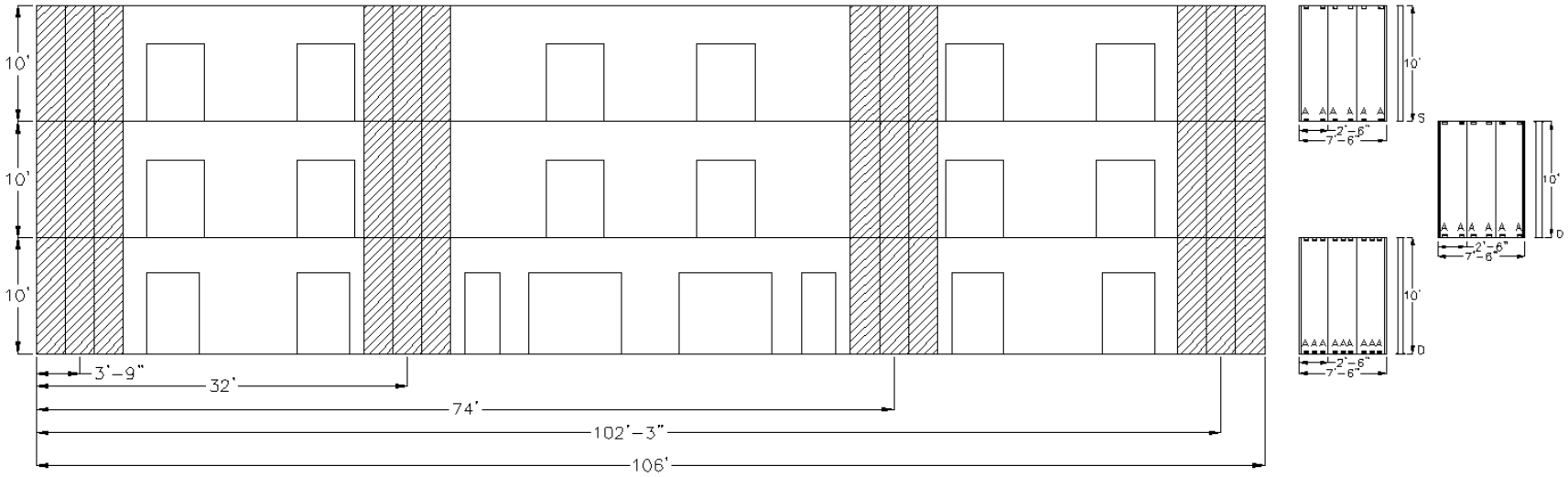


Figure B112. Archetype 99.

Table B100—Archetype 111 (7_A_3_2_ HR_LG_ DX_SP) design*, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.74	1.01	898.5	7.82	6.74	1.16
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/4	0.87	1.71	1785.7	15.63	13.39	1.17
1	3	M	2.5	6.875	5	A3	3	D	3126.0	2	1 3/4	0.78	2.38	2227.1	23.45	16.70	1.40

* There are four of (3)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

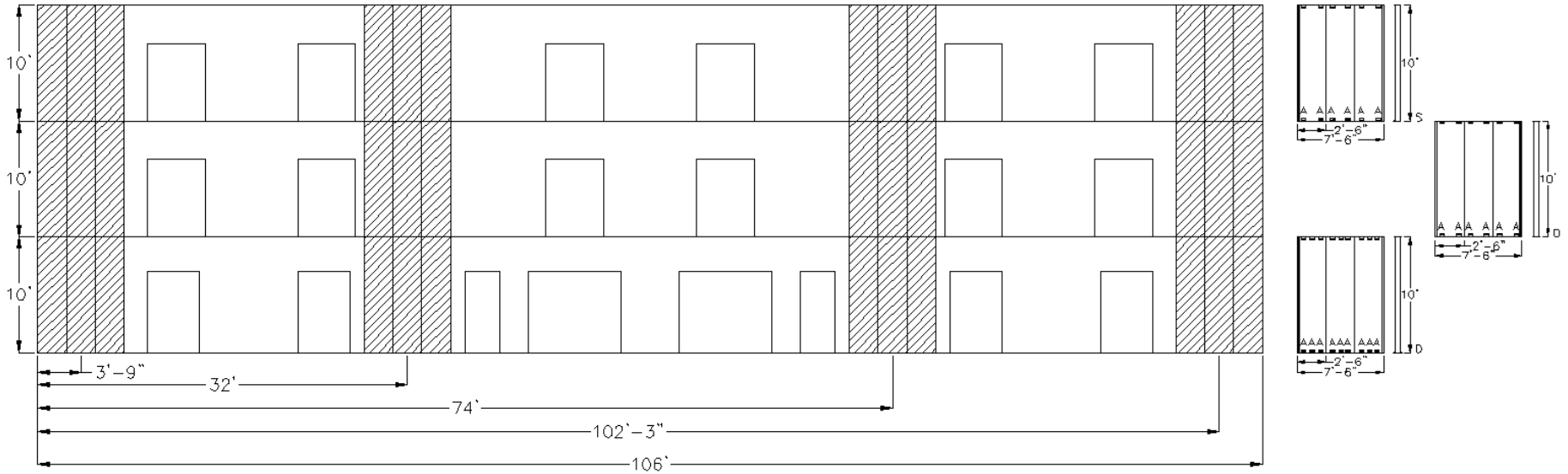


Figure B113. Archetype 111.

INPUT DATA

Total Height	h_n=	**30.0**	ft					
Total Weight	W=	**822**	k					
Seismic Design Category		**Dmax**						
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)					
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**		
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**		
	S_{DS} =	**1.000**	g ,					
	S_{D1} =	**0.600**	g					
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**						
The coefficient(ASCE Tab. 12.2.1)	R =	**4**						
	x =	**0.75**	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)					
	Cu=	**1.40**						
	T=Cu*Ta=	0.3589						
	Ts=	0.6						
	Cs=	0.25						
	k =	**1.00**	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	15,453						

DESIGN SUMMARY

Total base shear

V = 205.48

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	**208**	6,234	0.403	82.9		
	10.00						82.9	
2		20.0	**308**	6,156	0.398	81.9		829
	10.00						164.7	
1		10.0	**306**	3,063	0.198	40.7		2,476
	10.00						205.5	
		0.0						4,531

Seismic base shear calculation, low gravity, Index Building 7, 3-story, R = 4

INPUT DATA

Total Height	h_n=	**30.0**	ft					
Total Weight	W=	**975**	k					
Seismic Design Category		**Dmax**						
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)					
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**		
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**		
	S_{DS} =	**1.000**	g ,					
	S_{D1} =	**0.600**	g					
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**						
The coefficient(ASCE Tab. 12.2.1)	R =	**4**						
	x =	**0.75**	, (ASCE Tab 12.8-2)					
	$T_a = C_t(h_n)^x$ =	0.26	Sec, (ASCE 12.8.2.1)					
	Cu=	**1.40**						
	T=Cu*Ta=	0.3589						
	Ts=	0.6						
	Cs=	0.25						
	k =	**1.00**	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	17,745						

DESIGN SUMMARY

Total base shear

V = 243.68

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
3		30.0	**208**	6,234	0.351	85.6		
	10.00						85.6	
2		20.0	**384**	7,684	0.433	105.5		856
	10.00						191.1	
1		10.0	**383**	3,827	0.216	52.6		2,767
	10.00						243.7	
		0.0						5,204

Seismic base shear calculation, high gravity, Index Building 7, 3-story, R = 4

Table B101—Tributary load calculation, Index Building 7, 3-story, E-W direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 3	**A**	**1042.00**	**0.244**	82.89	**20.22**	85.61	**20.88**
	B	2180.00	0.510				
	C	1050.70	0.246				
Story 2	**A**	**1042.00**	**0.244**	164.75	**40.18**	191.12	**46.61**
	B	2180.00	0.510				
	C	1050.70	0.246				
Story 1	**A**	**1042.00**	**0.244**	205.48	**50.11**	243.68	**59.43**
	B	2180.00	0.510				
	C	1050.70	0.246				

Table B102—Archetype 99 (7_A_3_2_ HR_HG_ DX_SP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.63	0.93	695.9	7.82	5.22	1.50
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	1 1/8	0.87	1.49	1553.7	11.72	11.65	1.01
1	3	M	2.5	6.875	5	A3	2	D	2084.0	2	1 3/8	0.84	1.96	1980.9	15.63	14.86	1.05

* There are four of (3)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

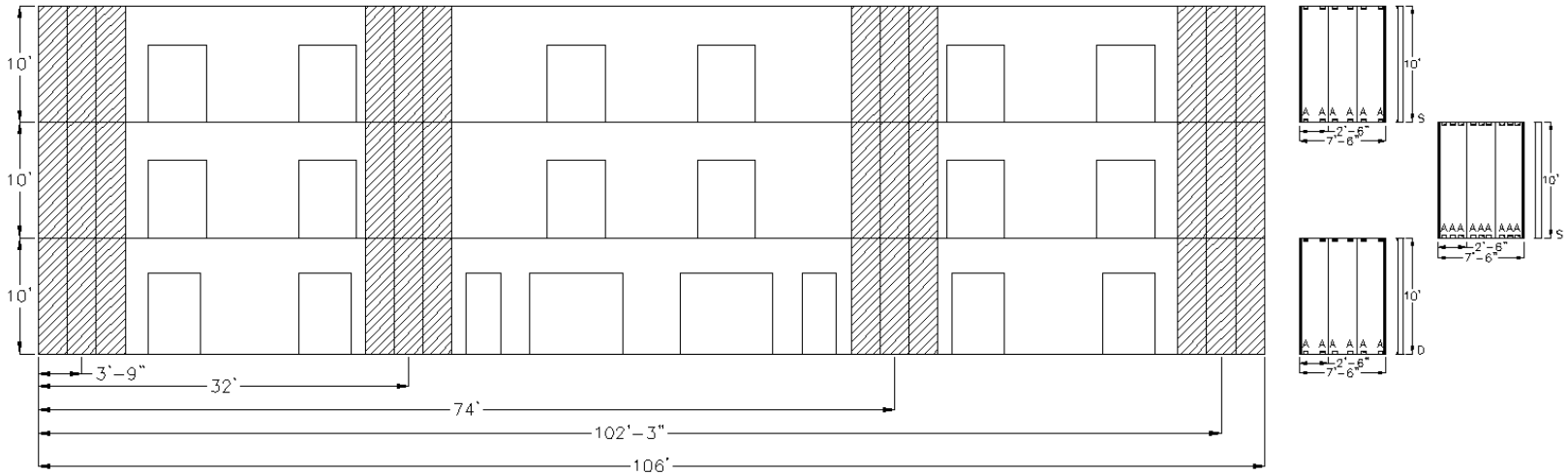


Figure B114. Archetype 99.

Table B103—Archetype 111 (7_A_3_2_ HR_LG_ DX_SP) design*, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)**	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.61	0.91	673.8	7.82	5.05	1.55
2	3	M	2.5	4.125	3	A3	3	S	1563.0	2	1	0.80	1.39	1339.2	11.72	10.04	1.17
1	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 1/2	0.87	1.63	1707.7	15.63	12.53	1.25

* There are four of (3)2.5ft walls along the wall line and all take equal load.
**Values shown are for unreinforced option and before applying C_d.

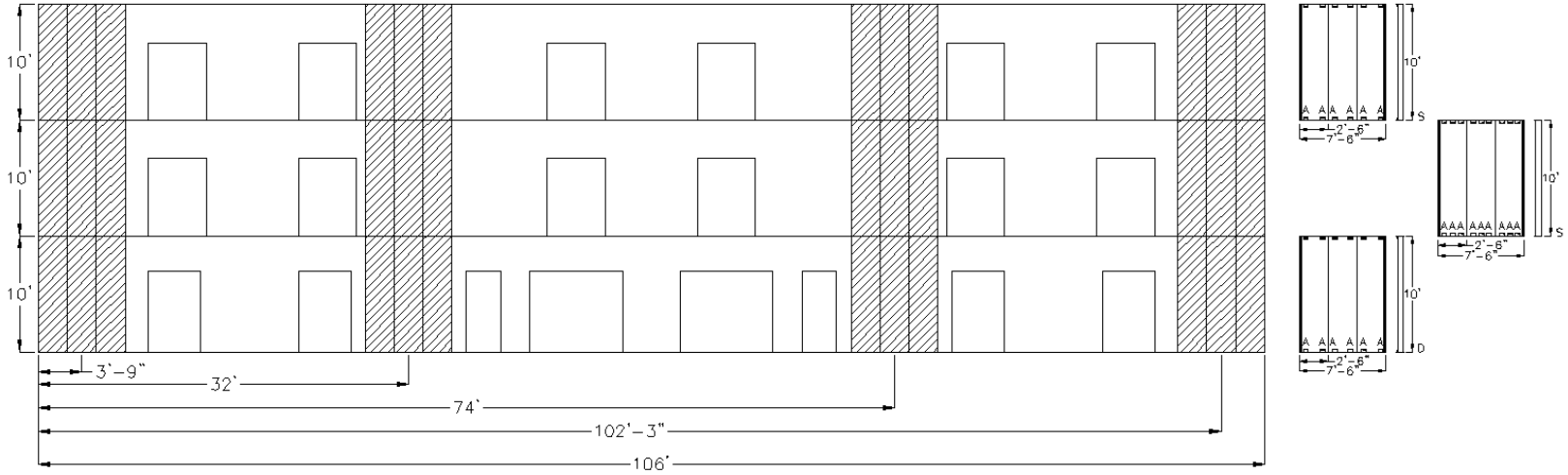


Figure B115. Archetype 111.

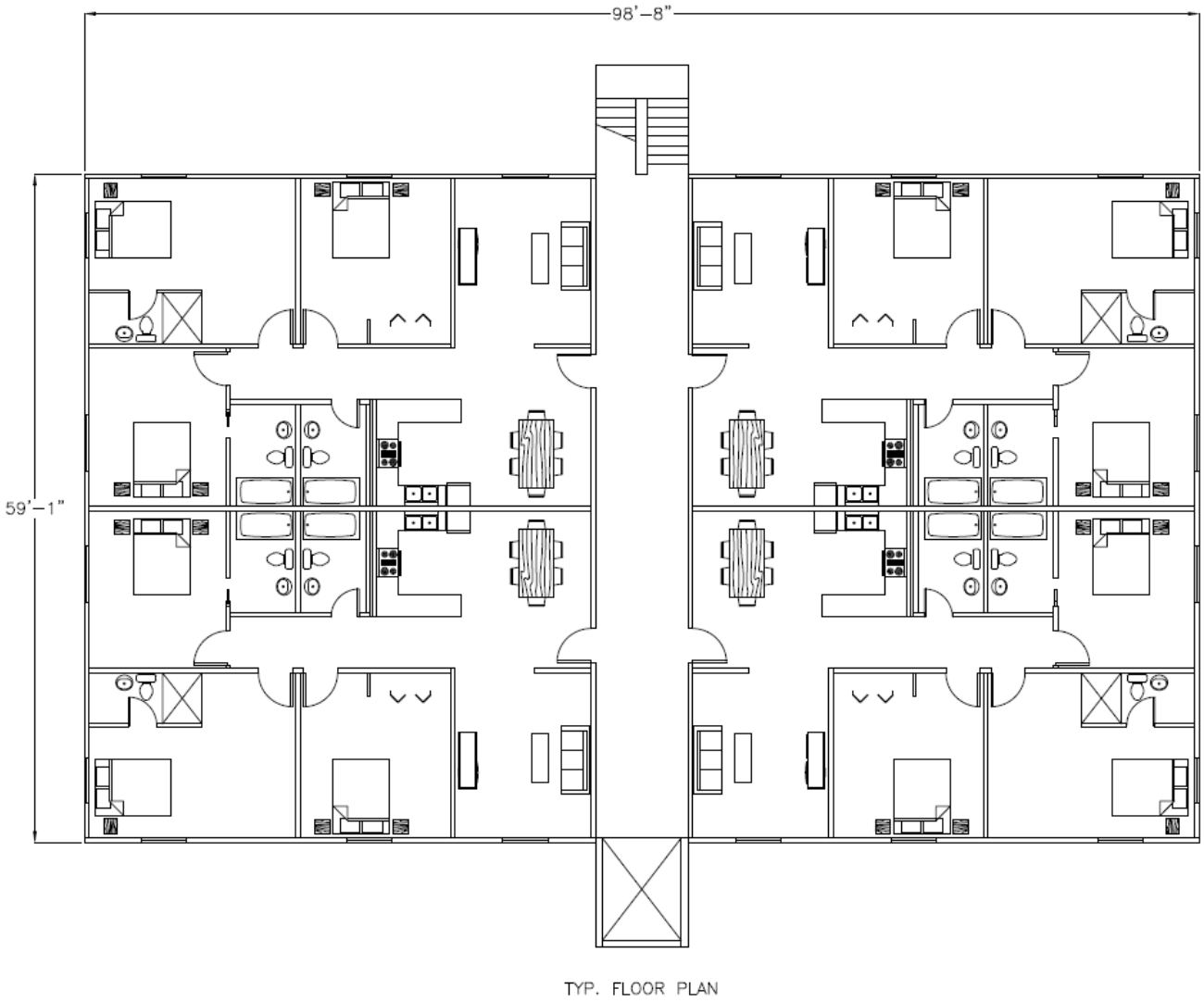


Figure B116. Index Building 8 floor plan.

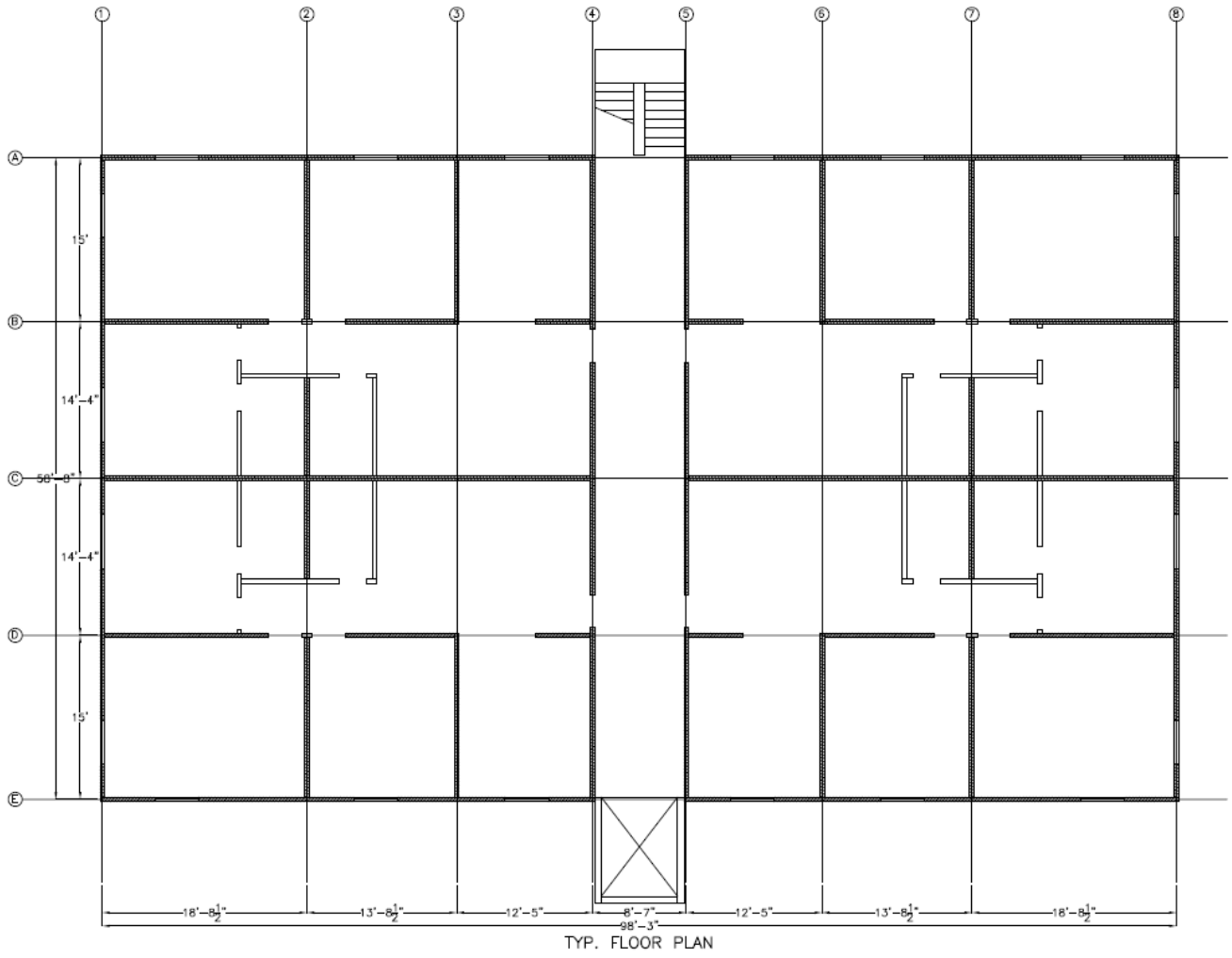


Figure B117. Index Building 8 assigned shear walls.

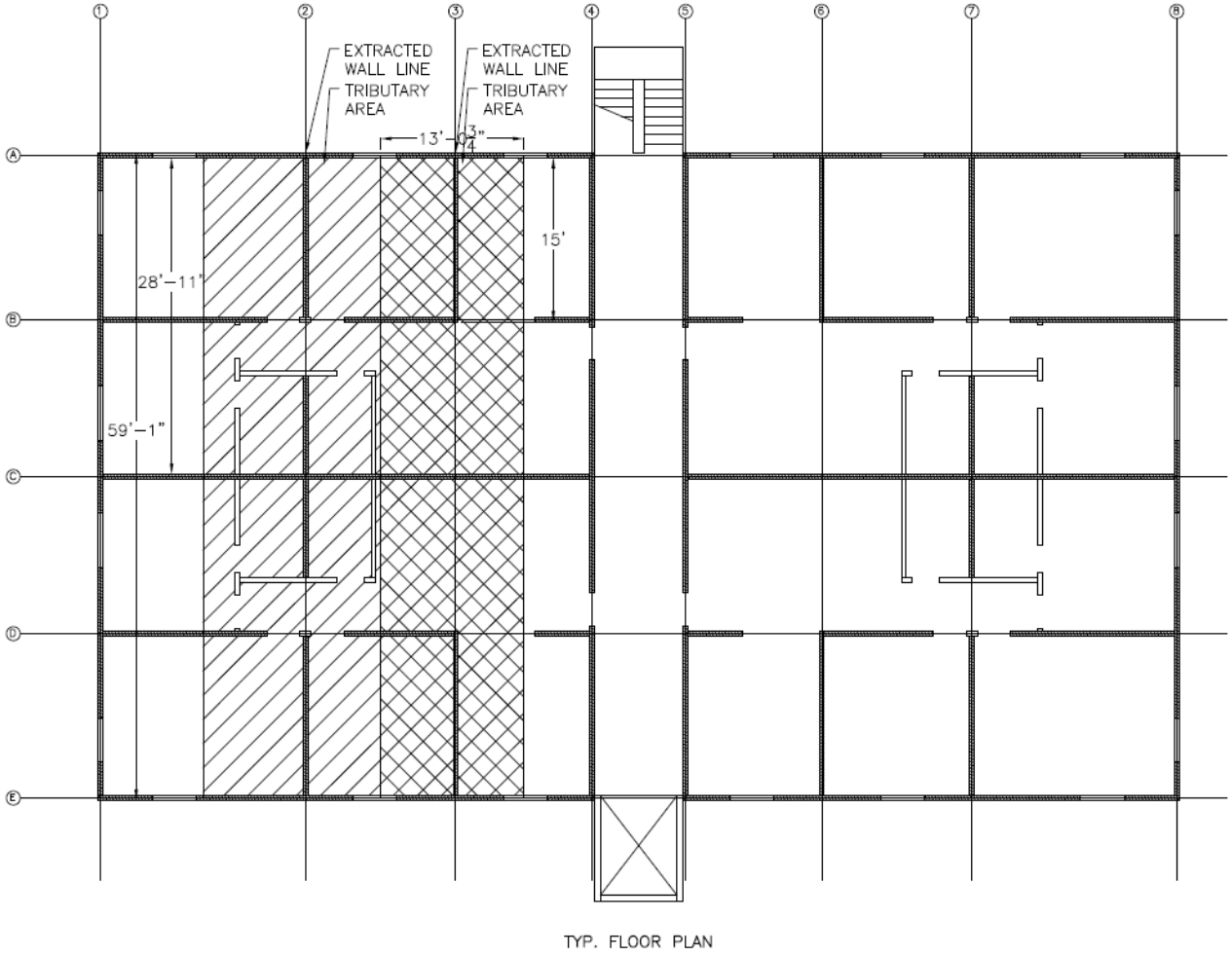


Figure B118. Index Building 8 tributary area.

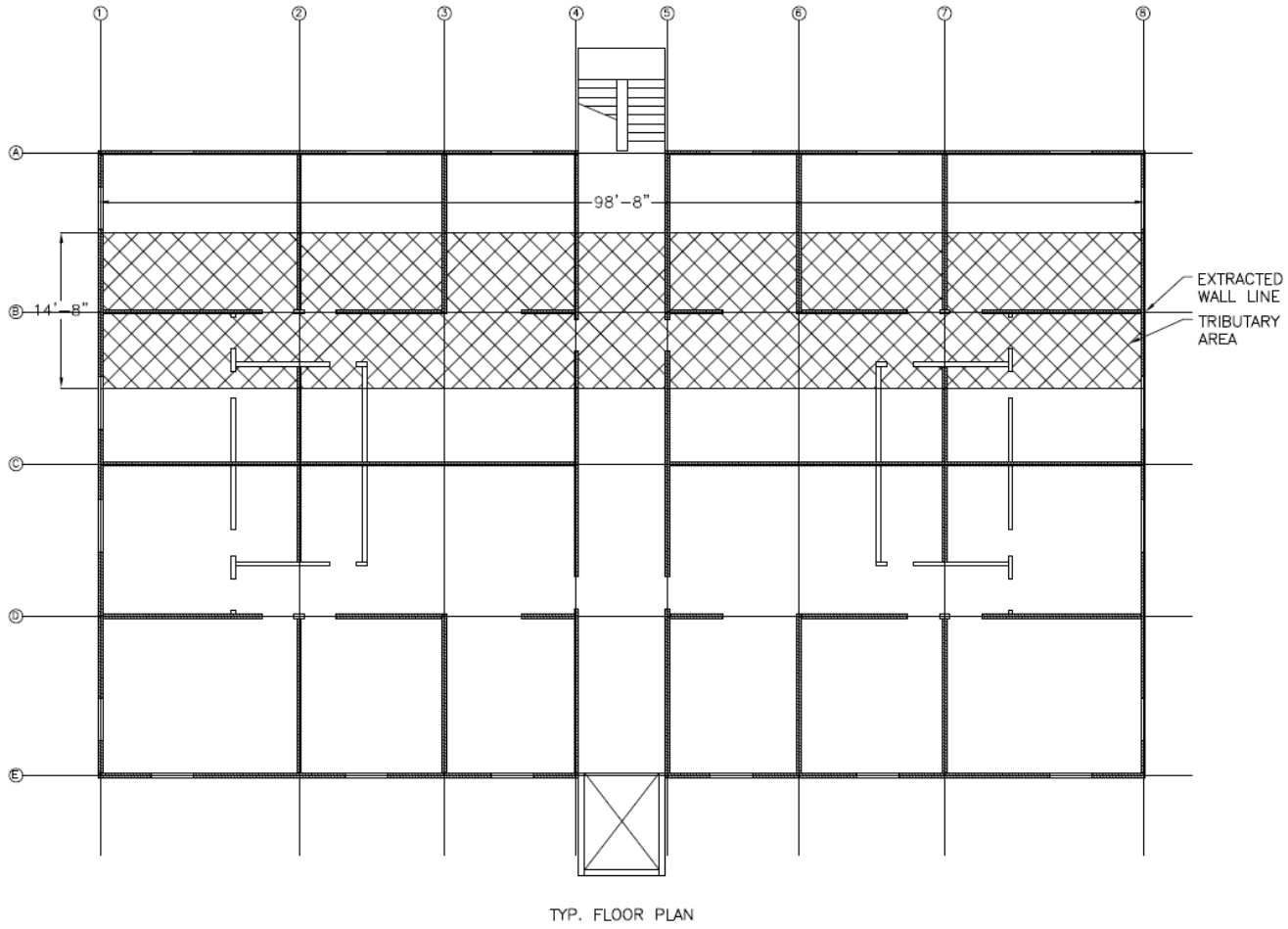


Figure B119. Index Building 8 tributary area.

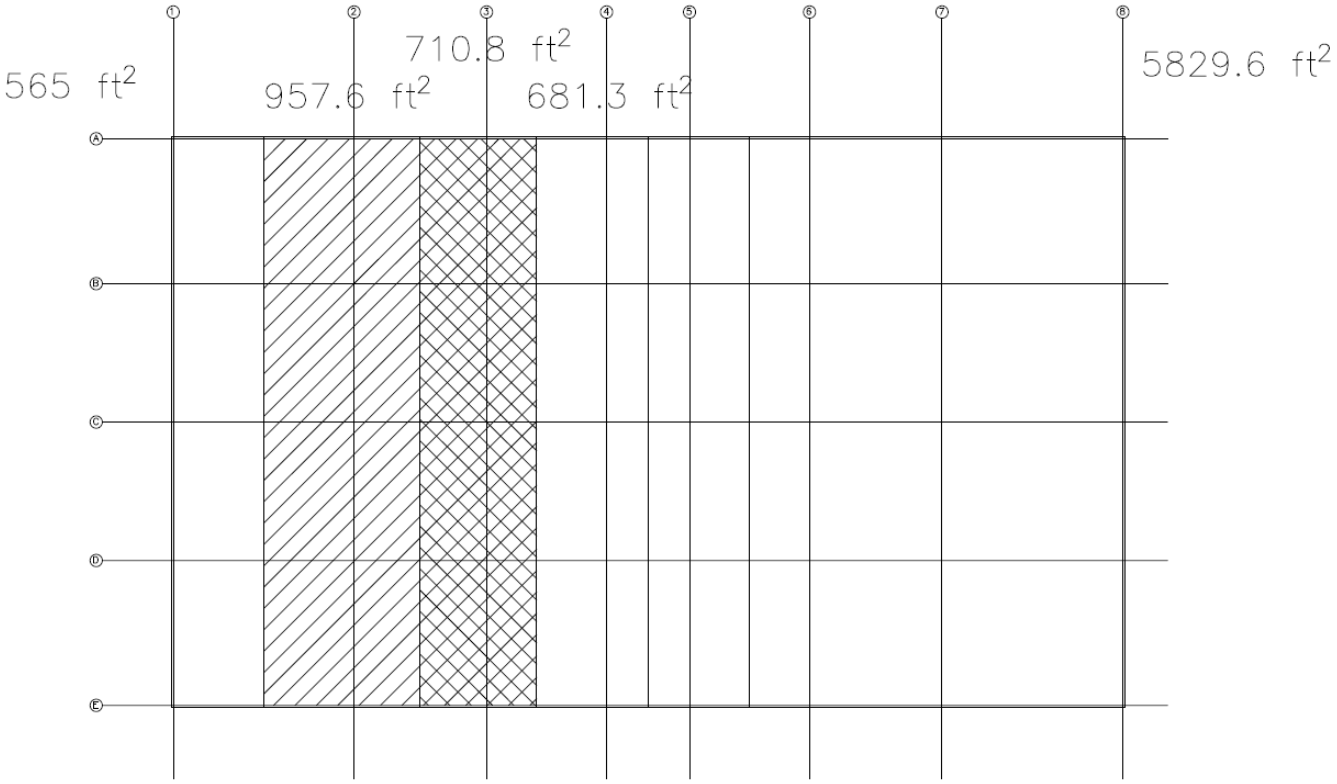


Figure B120. Index Building 8 tributary area.

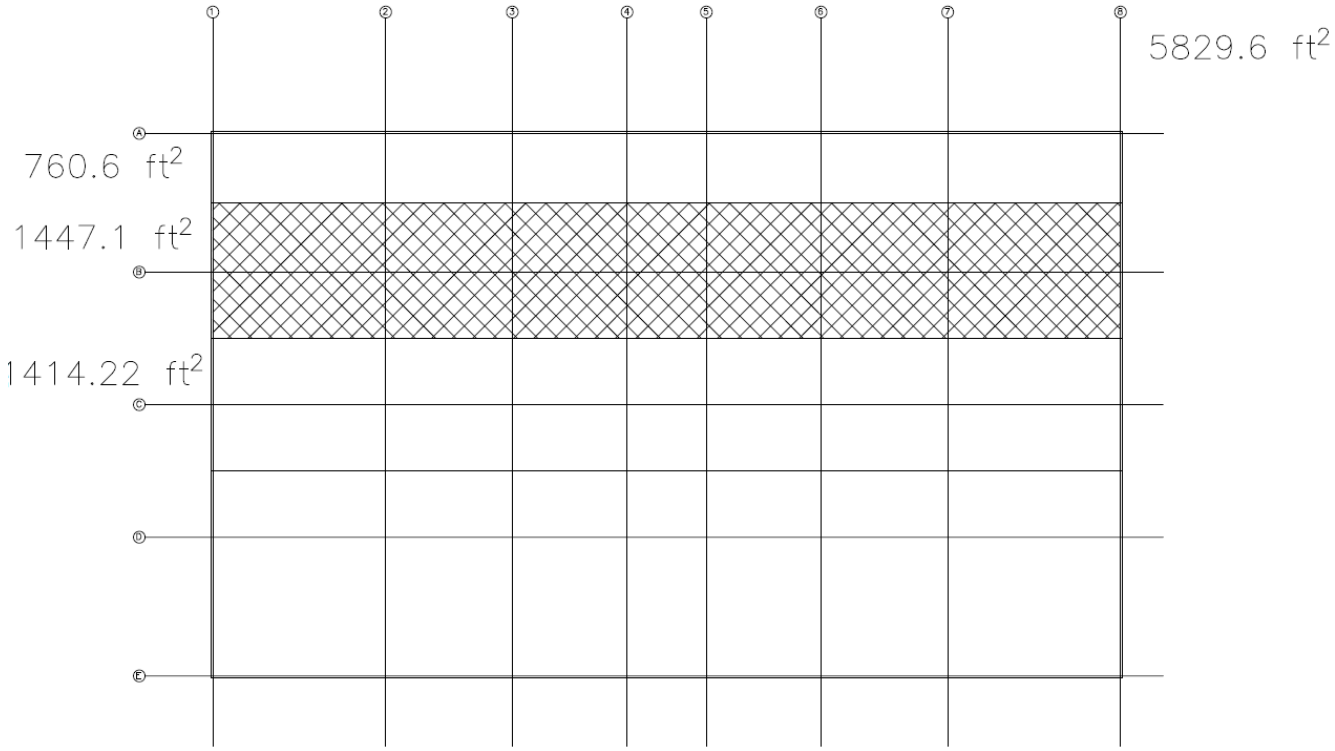


Figure B121. Index Building 8 tributary area.

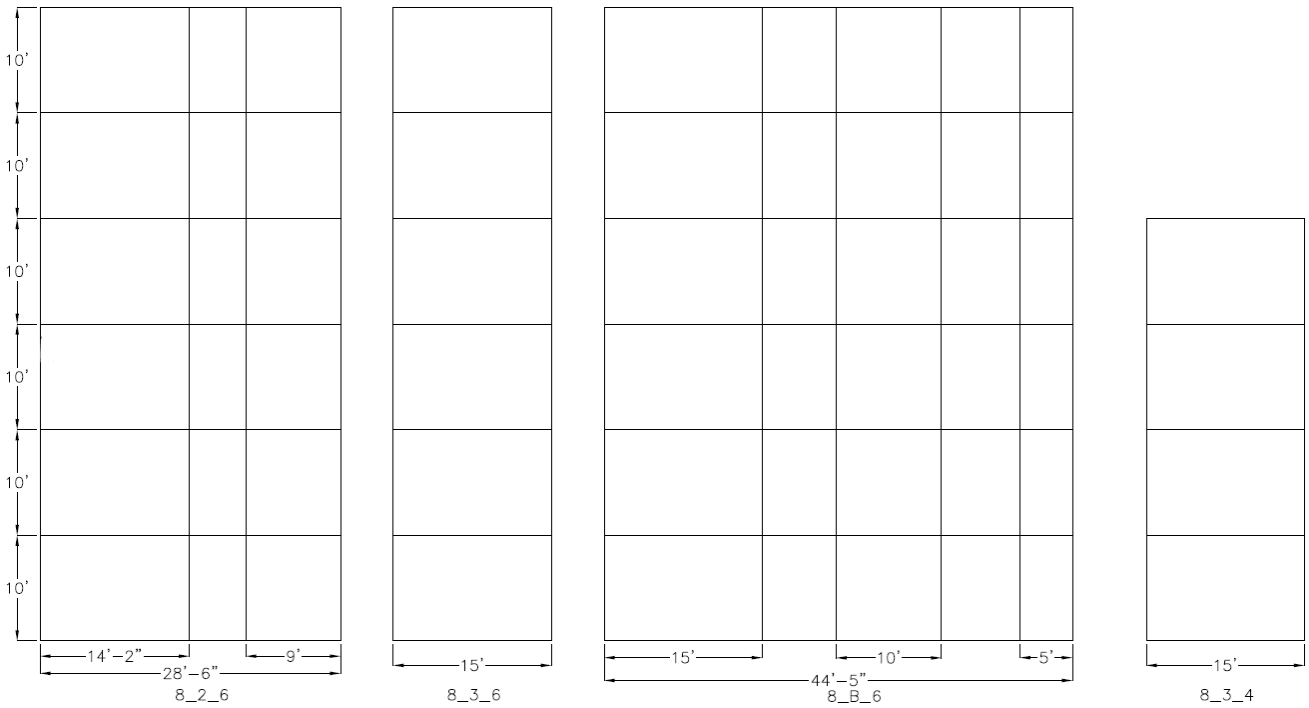


Figure B122. Index Building 8 extracted shear wall lines.

Table B104—Extracted archetypes, Index Building 8

Group No.		Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-2	2.5ft-20ft wall	Low aspect ratio panels	High	SDC D_{max}	8_3_6_1_ LR_HG_ DX_LP	5
PG-6			Low		8_3_6_1_ LR_LG_ DX_LP	17
PG-10		High aspect ratio panels	High		8_3_6_1_ HR_HG_ DX_LP	29
PG-14			Low		8_3_6_1_ HR_LG_ DX_LP	41
PG-17		Mixed aspect ratio	High		8_3_4_1_MR_HG_DX_SP	49
PG-18					8_3_6_1_ MR_HG_ DX_LP	53
PG-21			Low		8_3_4_1_MR_LG_DX_SP	61
PG-22					8_3_6_1_ MR_LG_ DX_LP	65
PG-26	20ft-60ft wall	Low aspect ratio panels	High		8_B_6_2_ LR_HG_ DX_LP	77
PG-30			Low		8_B_6_2_ LR_LG_ DX_LP	89
PG-34		High aspect ratio panels	High		8_B_6_2_ HR_HG_ DX_LP	101
PG-38			Low		8_B_6_2_ HR_LG_ DX_LP	113
PG-42		Mixed aspect ratio	High		8_B_6_2_ MR_HG_ DX_LP	125
PG-46			Low		8_B_6_2_ MR_LG_ DX_LP	137

Table B105—Seismic weight detailed weight calculation, Index Building 8, 4-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			5829.6					170613			294375	170613			294375
	4	10		314	2694	649.5	6042		90844	156681			90844	156681	
3			5829.6					182078			429602	287666			535191
	3	10		314	2694	649.5	6042		90844	156681			90844	156681	
2			5829.6					182078			429602	287666			535191
	2	10		314	2694	649.5	6042		90844	156681			90844	156681	
1			5829.6					182078			429602	287666			535191
	1	10		314	2694	649.5	6042		90844	156681			90844	156681	
Ground			0												

Table B106—Seismic weight detailed calculation, Index Building 8, 6-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{intwall}$	$A_{intWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{intWalls}$ (lb)	ΣW_{Level} (lb)
Roof			5829.6					170613			294375	170613			294375
	6	10		314	2694	649.5	6042		90844	156681			90844	156681	
5			5829.6					182078			429602	287666			535191
	5	10		314	2694	649.5	6042		90844	156681			90844	156681	
4			5829.6					182078			429602	287666			535191
	4	10		314	2694	649.5	6042		90844	156681			90844	156681	
3			5829.6					182078			429602	287666			535191
	3	10		314	2694	649.5	6042		90844	156681			90844	156681	
2			5829.6					182078			429602	287666			535191
	2	10		314	2694	649.5	6042		90844	156681			90844	156681	
1			5829.6					182078			429602	287666			535191
	1	10		314	2694	649.5	6042		90844	156681			90844	156681	
Ground			0												

Table B107—Seismic weight summary, Index Building 8, 4-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		294.4	294.4
	10		
3		429.6	535.2
	10		
2		429.6	535.2
	10		
1		429.6	535.2
	10		
Ground			

Table B108—Seismic weight summary, Index Building 8, 6-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		294.4	294.4
	10		
5		429.6	535.2
	10		
4		429.6	535.2
	10		
3		429.6	535.2
	10		
2		429.6	535.2
	10		
1		429.6	535.2
	10		
Ground			

INPUT DATA

Total Height	h_n=	40.0	ft				
Total Weight	W=	1,583	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.32	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.4454					
	Ts=	0.6					
	Cs=	0.25					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	37,552					

DESIGN SUMMARY

Total base shear V = 395.80

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
4		40.0	294	11,776	0.314	124.1		
	10.00						124.1	
3		30.0	430	12,888	0.343	135.8		1,241
	10.00						260.0	
2		20.0	430	8,592	0.229	90.6		3,841
	10.00						350.5	
1		10.0	430	4,296	0.114	45.3		7,346
	10.00						395.8	
		0.0						11,304

Seismic base shear calculation, low gravity, Index Building 8, 4-story, R = 3

INPUT DATA

Total Height	h_n=	40.0	ft				
Total Weight	W=	1,900	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.32	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.4454					
	Ts=	0.6					
	Cs=	0.25					
	k =	1.00	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	43,888					

DESIGN SUMMARY

Total base shear V = 475.00

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
4		40.0	294	11,776	0.268	127.5		
	10.00						127.5	
3		30.0	535	16,056	0.366	173.8		1,275
	10.00						301.2	
2		20.0	535	10,704	0.244	115.8		4,287
	10.00						417.1	
1		10.0	535	5,352	0.122	57.9		8,458
	10.00						475.0	
		0.0						13,208

Seismic base shear calculation, high gravity, Index Building 8, 4-story, R = 3

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	2,442	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^X$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	99,196					

DESIGN SUMMARY

Total base shear V = 809.24

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	294	21,839	0.220	178.2		
	10.00						178.2	
5		50.0	430	26,307	0.265	214.6		1,782
	10.00						392.8	
4		40.0	430	20,803	0.210	169.7		5,709
	10.00						562.5	
3		30.0	430	15,372	0.155	125.4		11,334
	10.00						687.9	
2		20.0	430	10,035	0.101	81.9		18,213
	10.00						769.7	
1		10.0	430	4,840	0.049	39.5		25,910
	10.00						809.2	
		0.0						34,003

Seismic base shear calculation, low gravity, Index Building 8, 6-story, R = 3

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	2,970	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^X$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	118,211					

DESIGN SUMMARY

Total base shear V = 984.18

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	294	21,839	0.185	181.8		
	10.00						181.8	
5		50.0	535	32,773	0.277	272.9		1,818
	10.00						454.7	
4		40.0	535	25,917	0.219	215.8		6,365
	10.00						670.5	
3		30.0	535	19,150	0.162	159.4		13,069
	10.00						829.9	
2		20.0	535	12,501	0.106	104.1		21,368
	10.00						934.0	
1		10.0	535	6,030	0.051	50.2		30,708
	10.00						984.2	
		0.0						40,550

Seismic base shear calculation, high gravity, Index Building 8, 6-story, R = 3

Table B109—Tributary load calculation, Index Building 8, 4-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 4	1	565.0	0.097	165.49		169.94	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**20.18**		**20.72**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 3	1	565.0	0.097	346.61		401.63	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**42.26**		**48.97**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 2	1	565.0	0.097	467.36		556.10	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**56.99**		**67.81**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 1	1	565.0	0.097	527.73		633.33	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**64.35**		**77.22**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				

Table B110—Tributary load calculation, Index Building 8, 4-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 4	A	760.6	0.13	165.49		169.94	
	B	**1447.1**	**0.25**		**41.08**		**42.18**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 3	A	760.6	0.13	346.61		401.63	
	B	**1447.1**	**0.25**		**86.04**		**99.70**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 2	A	760.6	0.13	467.36		556.10	
	B	**1447.1**	**0.25**		**116.01**		**138.04**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 1	A	760.6	0.13	527.73		633.33	
	B	**1447.1**	**0.25**		**131.00**		**157.21**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				

Table B111—Tributary load calculation, Index Building 8, 6-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft²)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	1	565.0	0.097	178.16		181.82	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**21.72**		**22.17**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 5	1	565.0	0.097	392.77		454.68	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**47.89**		**55.44**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 4	1	565.0	0.097	562.48		670.45	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**68.59**		**81.75**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 3	1	565.0	0.097	687.89		829.89	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**83.88**		**101.19**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 2	1	565.0	0.097	769.75		933.97	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**93.86**		**113.88**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 1	1	565.0	0.097	809.24		984.18	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**98.67**		**120.00**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B112—Tributary load calculation, Index Building 8, 6-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	760.6	0.13	178.16		181.82	
	B	**1447.1**	**0.25**		**44.22**		**45.13**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 5	A	760.6	0.13	392.77		454.68	
	B	**1447.1**	**0.25**		**97.50**		**112.87**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 4	A	760.6	0.13	562.48		670.45	
	B	**1447.1**	**0.25**		**139.63**		**166.43**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 3	A	760.6	0.13	687.89		829.89	
	B	**1447.1**	**0.25**		**170.76**		**206.01**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 2	A	760.6	0.13	769.75		933.97	
	B	**1447.1**	**0.25**		**191.08**		**231.84**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 1	A	760.6	0.13	809.24		984.18	
	B	**1447.1**	**0.25**		**200.88**		**244.30**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				

* Seismic load for shear wall line B is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B113—Archetype 5 (8_3_6_1_ LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.16	3.97	369.5	7.82	5.54	1.41
5	3	M	5	4.125	3	A3	4	S	1042.0	4	1/2	0.23	6.66	924.0	15.63	13.86	1.13
4	3	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.25	9.25	1362.5	23.45	20.44	1.15
3	3	M	5	4.125	3	A3	4	D	2084.0	4	1	0.25	11.38	1686.5	31.26	25.30	1.24
2	3	M	5	4.125	3	A3	4	D	2084.0	4	1 1/4	0.27	11.67	1898.0	31.26	28.47	1.10
1	3	M	5	4.125	3	A3	4	D	2084.0	4	1 3/8	0.29	11.48	2000.1	31.26	30.00	1.04

Table B114—Archetype 17 (8_3_6_1_ LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.15	3.98	362.1	7.82	5.43	1.44
5	3	M	5	4.125	3	A3	4	S	1042.0	4	5/8	0.19	7.10	798.2	15.63	11.97	1.31
4	3	M	5	4.125	3	A3	5	S	1302.5	4	3/4	0.23	8.20	1143.1	19.54	17.15	1.14
3	3	M	5	4.125	3	A3	6	S	1563.0	4	7/8	0.25	9.16	1397.9	23.45	20.97	1.12
2	3	M	5	4.125	3	A3	7	S	1823.5	4	1 1/8	0.25	10.40	1564.3	27.35	23.46	1.17
1	3	M	5	4.125	3	A3	7	S	1823.5	4	1 1/4	0.26	10.35	1644.6	27.35	24.67	1.11

*Values shown are for unreinforced option and before applying C_d.

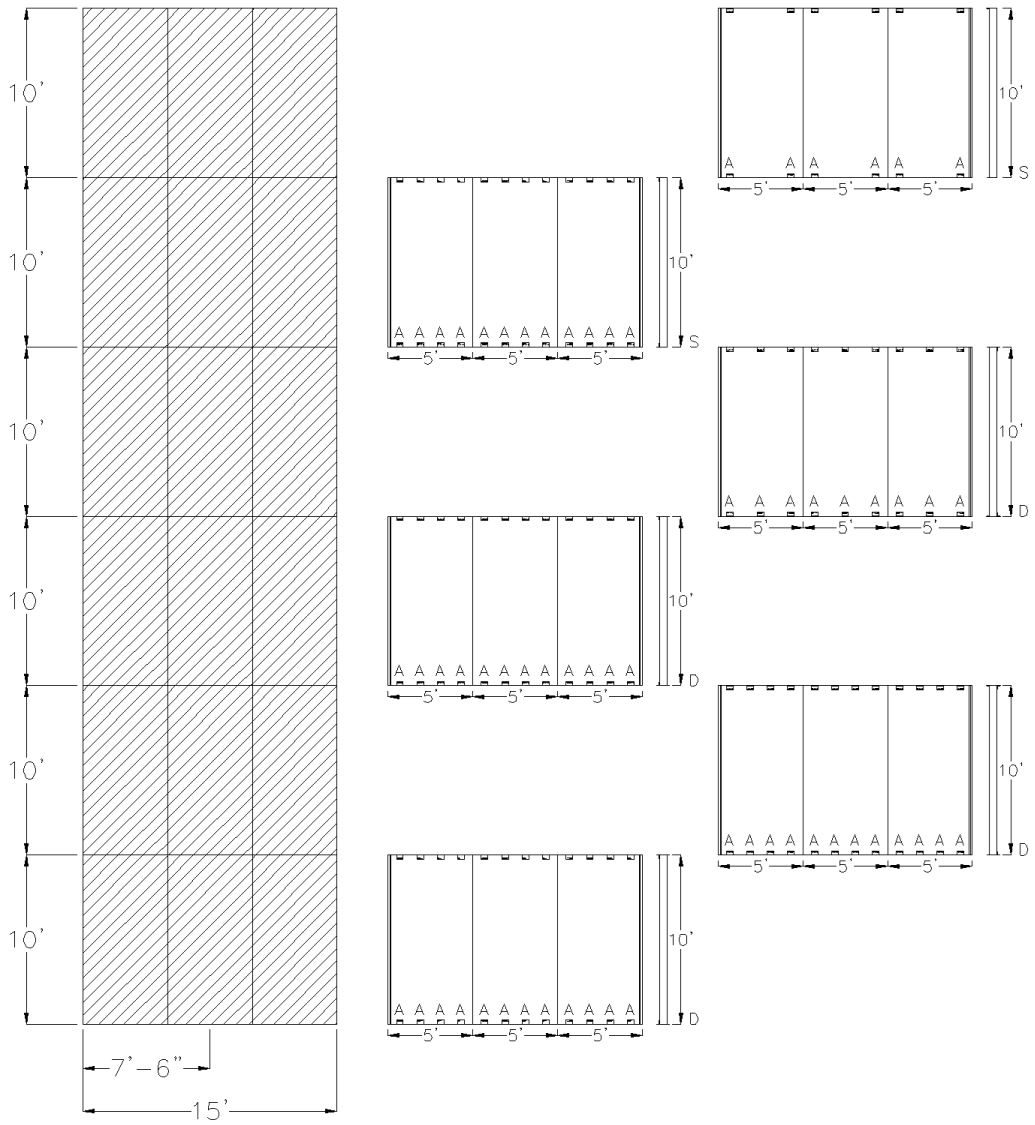


Figure B123. Archetype 5.

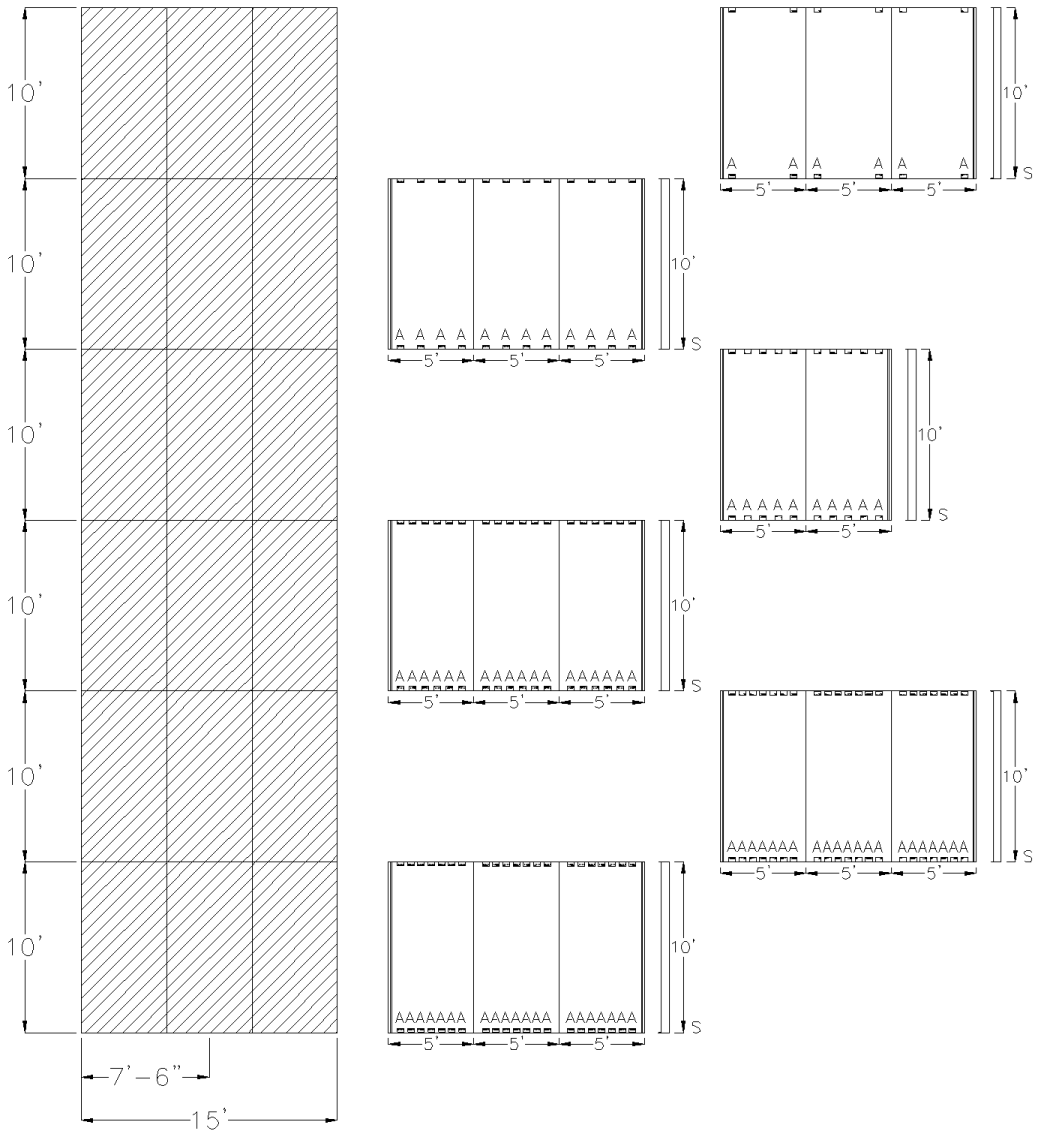


Figure B124. Archetype 17.

Table B115—Archetype 29 (8_3_6_1_ HR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	3.63	369.5	15.63	5.54	2.82
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.42	3.65	924.0	15.63	13.86	1.13
4	6	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.43	5.24	1362.5	31.26	20.44	1.53
3	6	M	2.5	4.125	3	A3	2	D	2084.0	4	1 1/8	0.53	5.31	1686.5	31.26	25.30	1.24
2	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.48	6.53	1898.0	31.26	28.47	1.10
1	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/2	0.51	6.51	2000.1	31.26	30.00	1.04

*Values shown are for unreinforced option and before applying C_d.

Table B116—Archetype 41 (8_3_6_1_ HR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	3.63	362.1	15.63	5.43	2.88
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.37	3.64	798.2	15.63	11.97	1.31
4	6	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.41	4.60	1143.1	23.45	17.15	1.37
3	6	M	2.5	6.875	5	A3	3	S	1563.0	4	1	0.43	5.43	1397.9	23.45	20.97	1.12
2	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.41	6.43	1564.3	31.26	23.46	1.33
1	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.43	6.40	1644.6	31.26	24.67	1.27

*Values shown are for unreinforced option and before applying C_d.

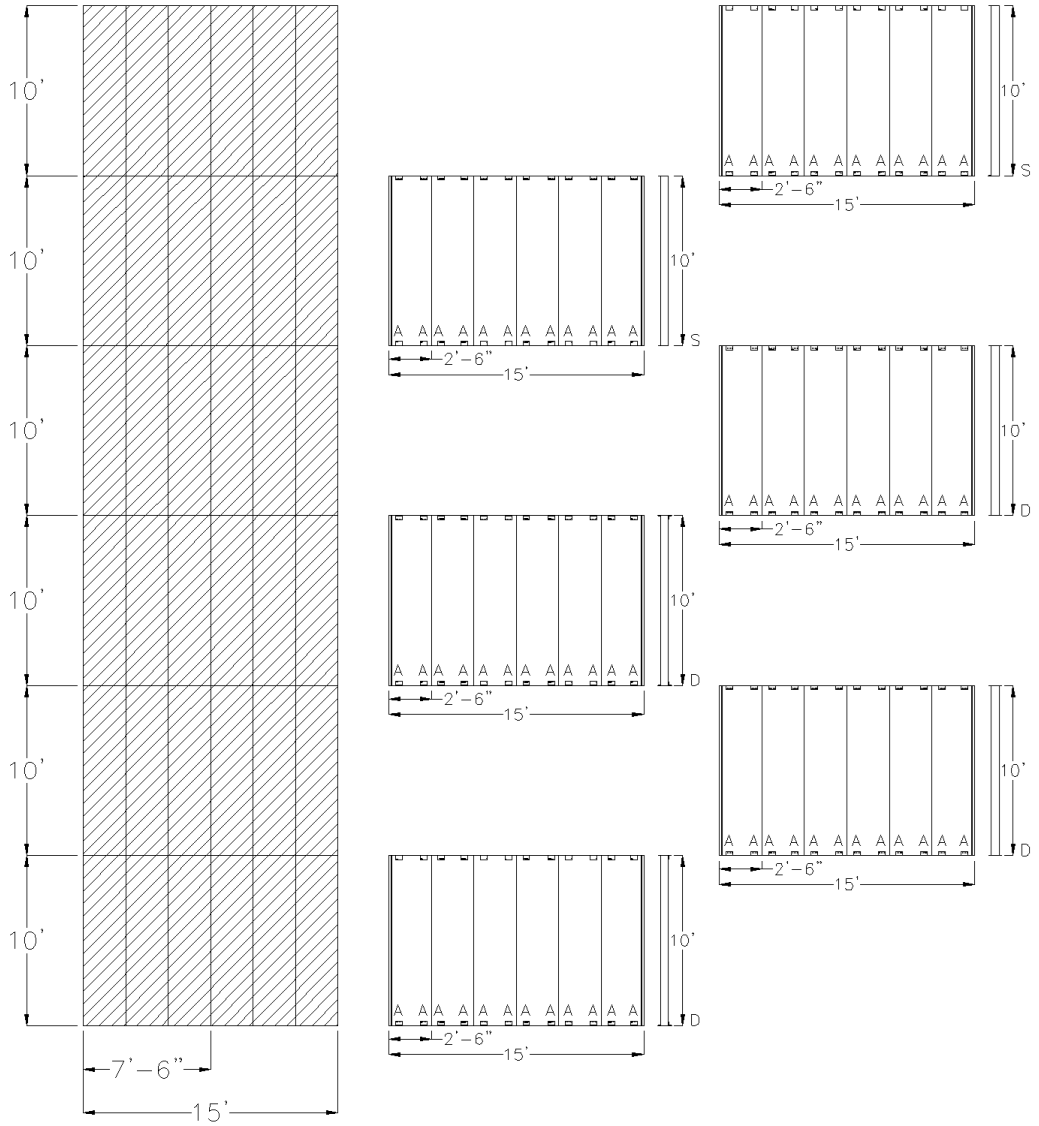


Figure B125. Archetype 29.

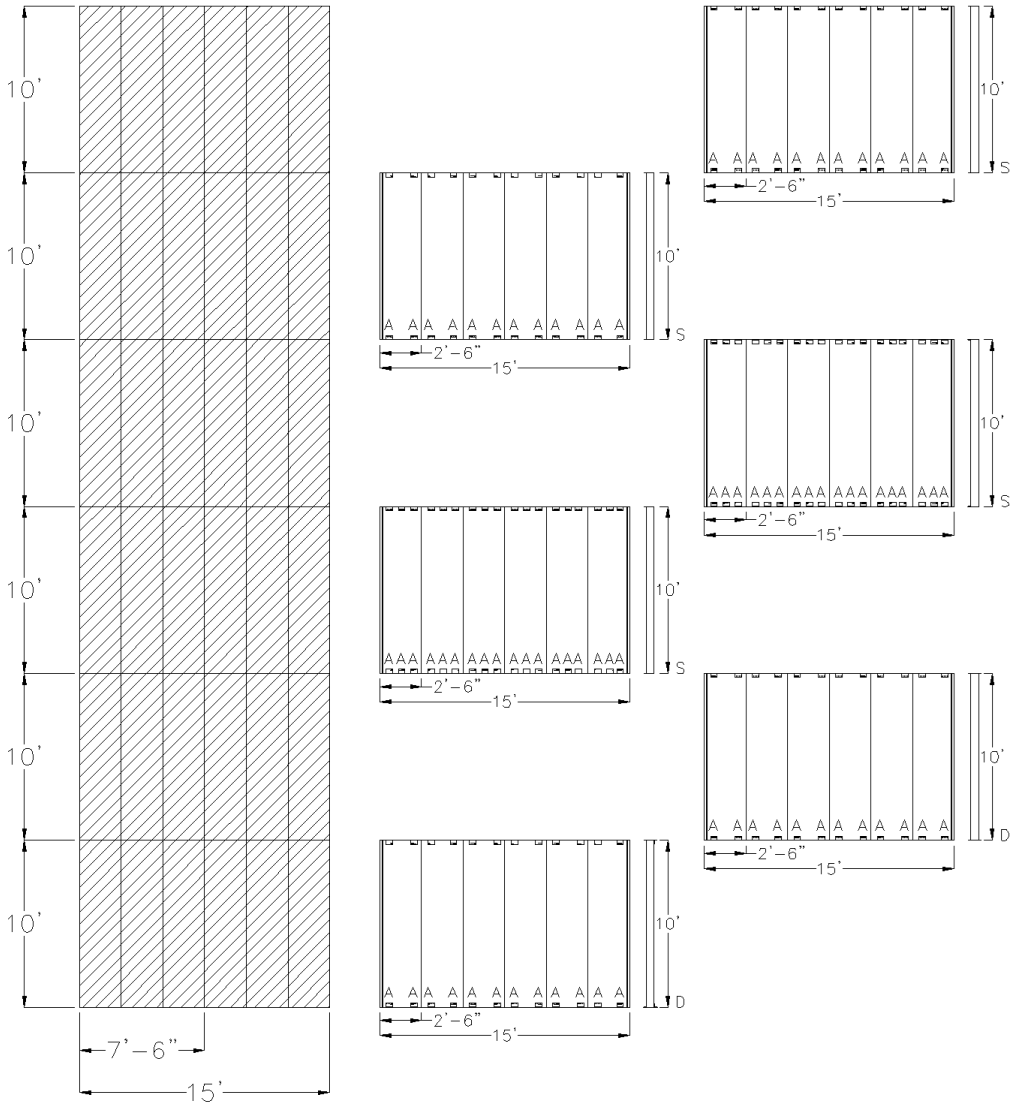


Figure B126. Archetype 41.

Table B117—Archetype 53 (8_3_6_1_ MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.18	0.99	316.5	-	-	-
5	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.25	1.04	468.6	-	-	-
4	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.28	1.06	533.7	-	-	-
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.31	1.04	575.5	-	-	-
2	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 3/8	0.27	1.14	562.9	-	-	-
1	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.28	1.14	564.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.18	2.47	396.0	10.42	5.54	1.88
5	2	M	5	4.125	3	A3	5	S	1302.5	4	5/8	0.25	5.11	1151.7	18.24	13.86	1.32
4	2	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.28	7.05	1776.9	26.05	20.44	1.27
3	2	M	5	4.125	3	A3	5	D	2605.0	4	1 1/8	0.31	8.11	2242.0	31.26	25.30	1.24
2	2	M	5	6.875	5	A3	6	D	3126.0	4	1 3/8	0.27	10.40	2565.6	36.47	28.47	1.28
1	2	M	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.28	10.94	2718.0	36.47	30.00	1.22

*Values shown are for reinforced option and before applying C_d.

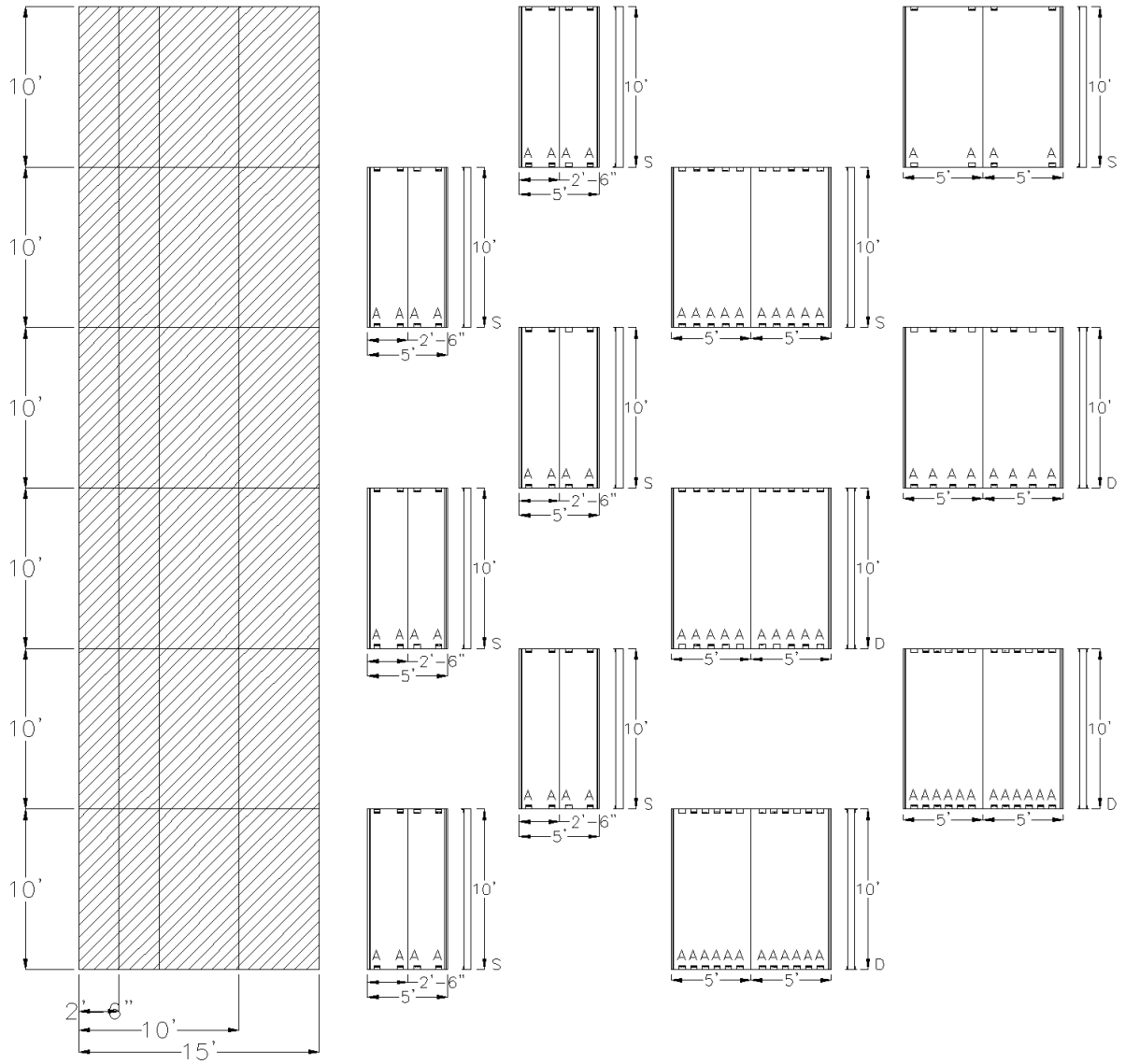


Figure B127. Archetype 53.

Table B118—Archetype 65 (8_3_6_1_ MR_LG_ DX_LP design), R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.41	1.26	464.1	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.62	1.59	882.3	-	-	-
4	4	M	2.5	6.875	5	A3	3	S	1563.0	4	7/8	0.64	2.43	1402.8	-	-	-
3	4	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.67	2.90	1756.3	-	-	-
2	4	M	2.5	9.625	7	A3	2	D	2084.0	4	1 3/8	0.67	3.28	1986.9	-	-	-
1	4	M	2.5	12.375	7	A3	2	D	2084.0	4	1 1/2	0.66	3.50	2073.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	2	S	521.0	1	1/2	0.41	0.21	157.9	13.03	5.43	2.40
5	1	S	5	4.125	3	A3	3	S	781.5	2	1/2	0.62	0.57	629.9	14.33	11.97	1.20
4	1	S	5	6.875	5	A3	3	S	781.5	2	5/8	0.64	0.54	623.6	19.54	17.15	1.14
3	1	S	5	6.875	5	A3	3	S	781.5	2	3/4	0.67	0.56	681.3	24.75	20.97	1.18
2	1	S	5	9.625	7	A3	3	S	781.5	2	7/8	0.67	0.59	719.1	24.75	23.46	1.05
1	1	S	5	12.375	7	A3	4	S	1042.0	2	1	0.66	0.66	787.3	26.05	24.67	1.06

*Values shown are for unreinforced option and before applying C_d.

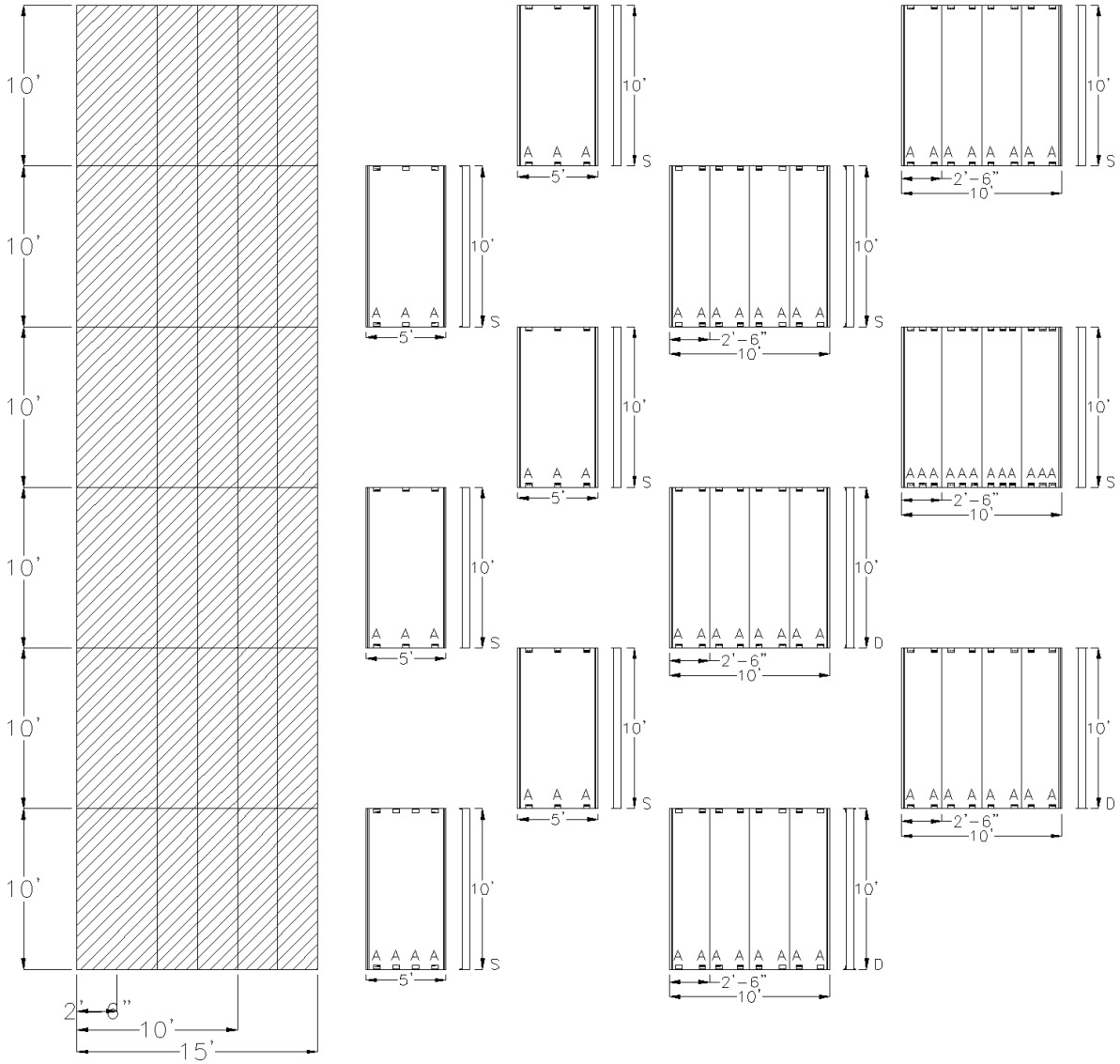


Figure B128. Archetype 65.

Table B119—Archetype 49 (8_3_4_1_MR_HG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length	Thickness (in.)	No. of Layers	Connector type	NC, Number of	S/D	Shear Capacity	No. of	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load	Shear Strength	Story Shear (kip)/Archetype	Ratio of Provided
4	3	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.62	0.98	727.7	-	-	-
3	3	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.81	1.68	1633.0	-	-	-
2	3	M	2.5	6.875	5	A3	3	D	3126.0	4	1	0.81	2.48	2413.1	-	-	-
1	3	M	2.5	12.375	7	A3	3	D	3126.0	4	1	0.76	3.09	2828.3	-	-	-
Story	No. of Panels	Configuration	Panel Length	Thickness (in.)	No. of Layers	Connector type	NC, Number of	S/D	Shear Capacity	No. of	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load	Shear Strength	Story Shear (kip)/Archetype	Ratio of Provided
4	1	S	5	4.125	3	A3	4	S	1042.0	4	1/2	0.62	0.88	980.5	13.03	10.36	1.26
3	1	S	5	4.125	3	A3	5	D	2605.0	4	7/8	0.81	1.68	2447.8	28.66	24.49	1.17
2	1	S	5	6.875	5	A3	7	D	3647.0	4	1	0.81	2.16	3161.1	41.68	33.90	1.23
1	1	S	5	12.375	7	A3	7	D	3647.0	4	1	0.76	2.53	3480.1	41.68	38.61	1.08

*Values shown are for unreinforced option and before applying C_d.

Table B120—Archetype 61 (8_3_4_1_MR_LG_DX_SP) design, R = 3

Story	No. of Panels	Configuration	Panel Length	Thickness (in.)	No. of Layers	Connector type	NC, Number of	S/D	Shear Capacity	No. of	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load	Shear Strength	Story Shear (kip)/Archetype	Ratio of Provided
4	3	M	2.5	4.125	3	A3	2	S	1042.0	2	3/4	0.33	1.71	684.5	-	-	-
3	3	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.45	1.65	885.8	-	-	-
2	3	M	2.5	4.125	3	A3	2	D	2084.0	2	1 3/8	0.62	2.51	1850.8	-	-	-
1	3	M	2.5	6.875	5	A3	2	D	2084.0	2	2	0.50	3.15	1899.7	-	-	-
Story	No. of Panels	Configuration	Panel Length	Thickness (in.)	No. of Layers	Connector type	NC, Number of	S/D	Shear Capacity	No. of	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load	Shear Strength	Story Shear (kip)/Archetype	Ratio of Provided
4	1	S	5	4.125	3	A3	4	S	1042.0	2	1/2	0.33	1.65	991.1	13.03	10.09	1.29
3	1	S	5	4.125	3	A3	6	D	3126.0	4	3/4	0.45	3.60	2897.7	23.45	21.13	1.11
2	1	S	5	4.125	3	A3	6	D	3126.0	4	7/8	0.62	2.64	2922.5	31.26	28.49	1.10
1	1	S	5	6.875	5	A3	7	D	3647.0	4	1 1/8	0.50	3.96	3585.3	33.87	32.17	1.05

*Values shown are for reinforced option and before applying C_d.

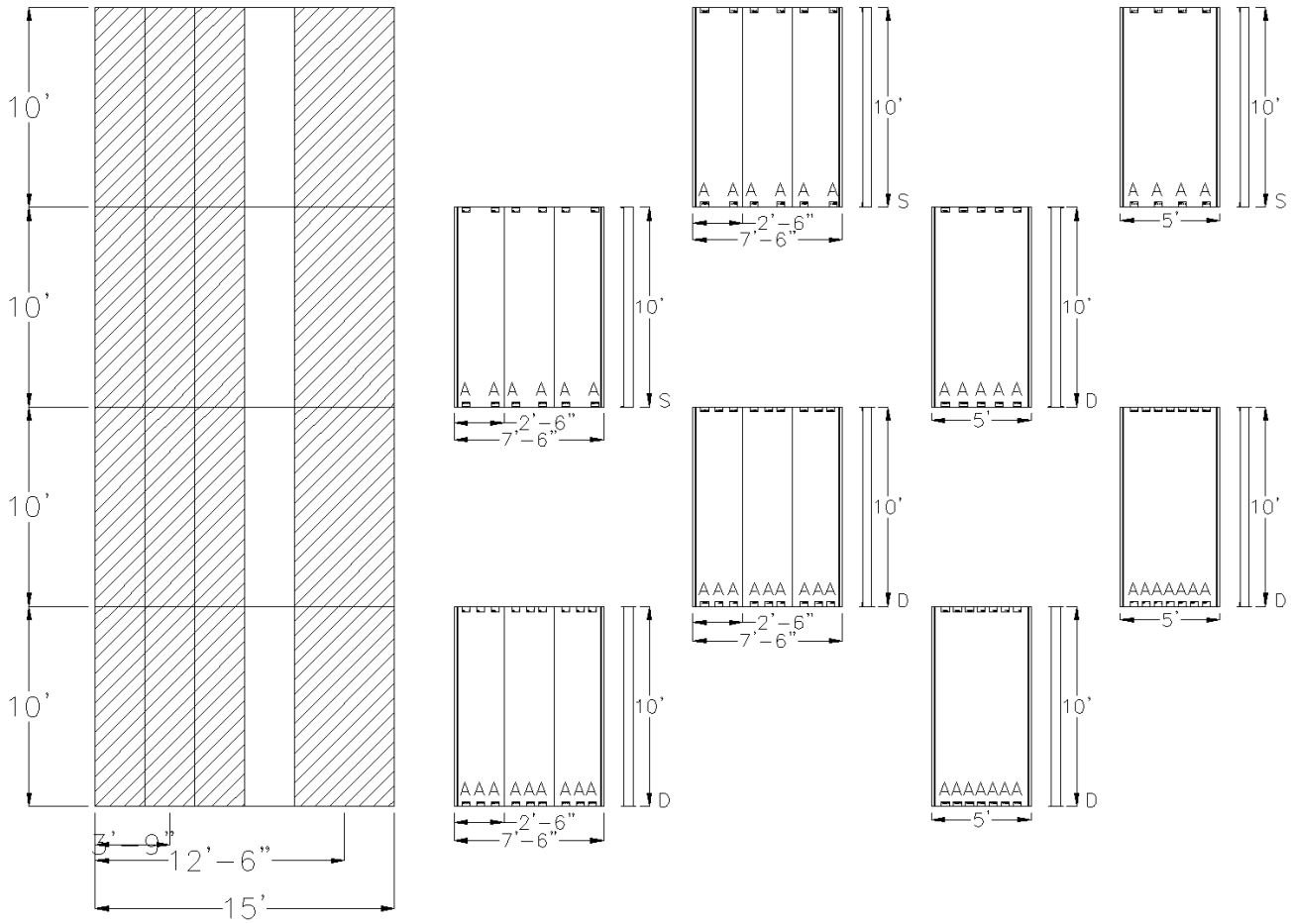


Figure B129. Archetype 49.

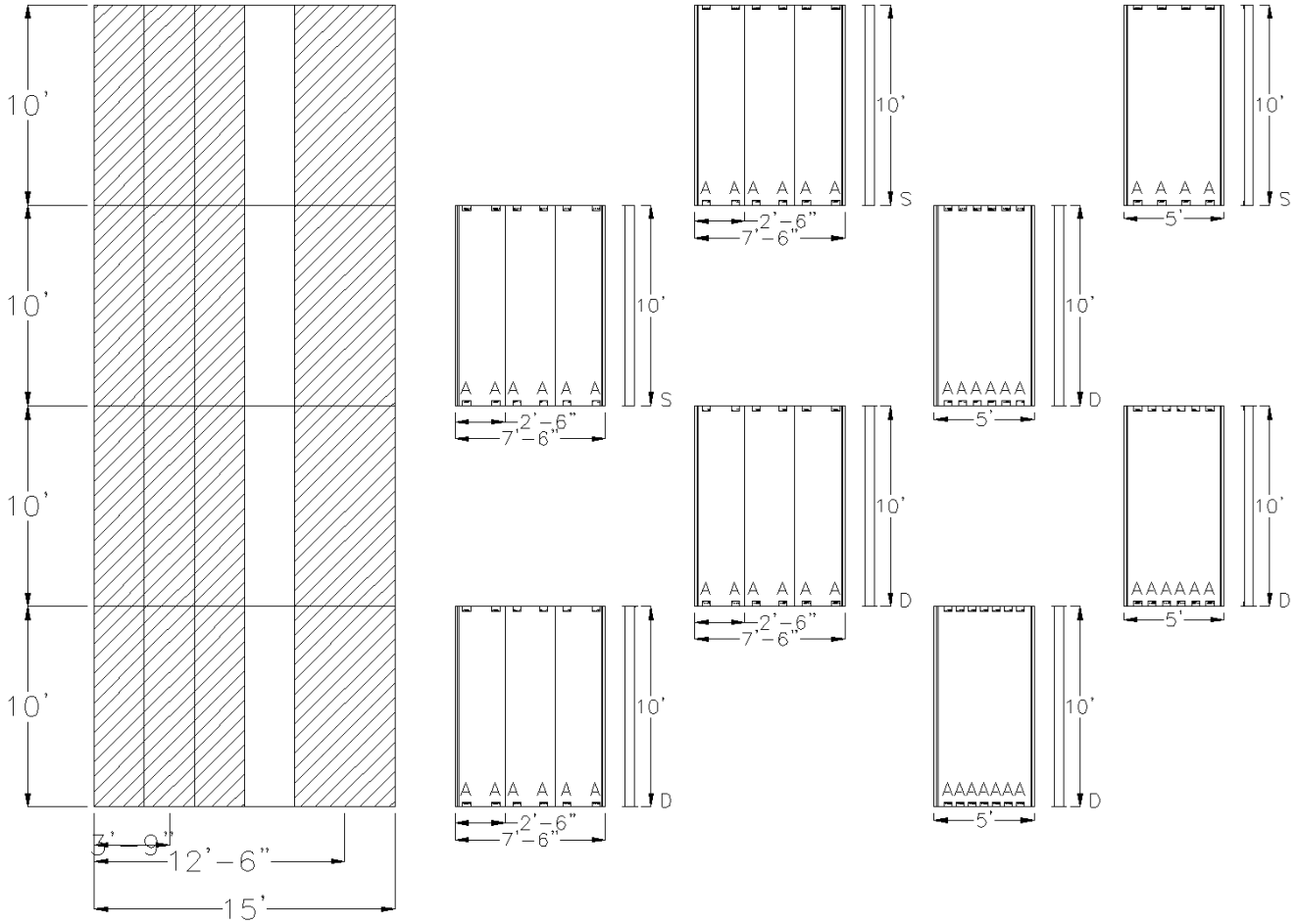


Figure B130. Archetype 61.

Table B121—Archetype 77 (8_B_6_2_ LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.13	3	A3	2	S	521.0	4	3/8	0.17	3.77	380.2	-	-	-
5	3	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.23	9.52	1304.3	-	-	-
4	3	M	5	4.125	3	A3	5	D	2605.0	4	1	0.26	13.14	2036.2	-	-	-
3	3	M	5	6.875	5	A3	5	D	2605.0	4	1 1/4	0.26	15.08	2368.0	-	-	-
2	3	M	5	6.875	5	A3	6	D	3126.0	4	1 1/2	0.27	17.13	2769.1	-	-	-
1	3	M	5	6.875	5	A3	7	D	3647.0	4	1 3/4	0.27	18.91	3066.5	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.17	2.32	351.5	-	-	-
5	2	M	5	4.125	3	A3	3	S	781.5	4	1/2	0.23	3.09	634.7	-	-	-
4	2	M	5	4.125	3	A3	4	S	1042.0	4	3/4	0.26	4.24	984.2	-	-	-
3	2	M	5	6.875	5	A3	3	D	1563.0	4	7/8	0.26	5.93	1397.2	-	-	-
2	2	M	5	6.875	5	A3	3	D	1563.0	4	1	0.27	5.79	1402.9	-	-	-
1	2	M	5	6.875	5	A3	3	D	1563.0	4	1 1/8	0.27	5.71	1387.7	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	2	1/2	0.17	1.41	426.2	16.93	11.35	1.49
5	1	S	5	4.125	3	A3	3	S	781.5	2	5/8	0.23	1.25	512.4	35.17	28.47	1.24
4	1	S	5	4.125	3	A3	3	S	781.5	2	5/8	0.26	0.70	326.2	53.40	42.02	1.27
3	1	S	5	6.875	5	A3	3	S	781.5	2	3/4	0.26	1.07	506.2	58.61	52.02	1.13
2	1	S	5	6.875	5	A3	3	S	781.5	2	7/8	0.27	1.23	597.8	66.43	58.55	1.13
1	1	S	5	6.875	5	A3	3	S	781.5	2	7/8	0.27	0.75	366.1	74.24	61.71	1.20

*Values shown are for reinforced option and before applying C_d.

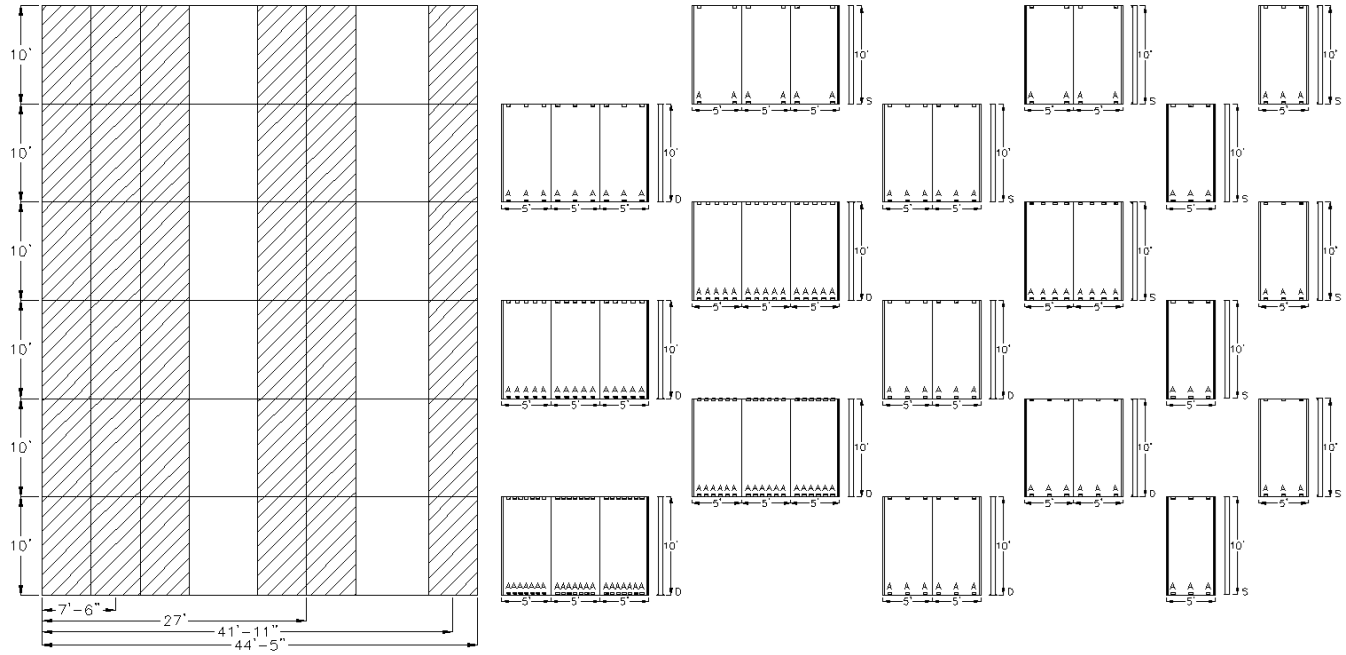


Figure B131. Archetype 77.

Table B122—Archetype 89 (8_B_6_2_ LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.17	3.77	377.0	-	-	-
5	3	M	5	4.125	3	A3	5	S	1302.5	4	5/8	0.22	7.93	1059.9	-	-	-
4	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.27	11.20	1786.2	-	-	-
3	3	M	5	4.125	3	A3	5	D	2605.0	4	1 1/8	0.27	12.89	2101.5	-	-	-
2	3	M	5	4.125	3	A3	6	D	3126.0	4	1 3/8	0.28	14.43	2393.9	-	-	-
1	3	M	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.25	17.38	2641.8	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.17	2.32	348.7	-	-	-
5	2	M	5	4.125	3	A3	3	S	781.5	4	1/2	0.22	3.09	618.5	-	-	-
4	2	M	5	4.125	3	A3	3	S	781.5	4	5/8	0.27	3.13	748.9	-	-	-
3	2	M	5	4.125	3	A3	4	S	1042.0	4	3/4	0.27	3.88	948.1	-	-	-
2	2	M	5	4.125	3	A3	4	S	1042.0	4	7/8	0.28	3.89	967.7	-	-	-
1	2	M	5	6.875	5	A3	4	S	1042.0	4	1	0.25	4.13	941.9	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	2	S	521.0	2	1/2	0.17	1.28	382.8	15.63	11.06	1.41
5	1	S	5	4.125	3	A3	2	S	521.0	2	5/8	0.22	1.14	458.1	29.96	24.37	1.23
4	1	S	5	4.125	3	A3	2	S	521.0	2	1/2	0.27	0.26	124.9	41.68	34.91	1.19
3	1	S	5	4.125	3	A3	2	S	521.0	2	5/8	0.27	0.69	337.2	52.10	42.69	1.22
2	1	S	5	4.125	3	A3	2	S	521.0	2	3/4	0.28	0.88	436.7	59.92	47.77	1.25
1	1	S	5	6.875	5	A3	2	S	521.0	2	3/4	0.25	0.51	234.6	59.92	50.22	1.19

*Values shown are for reinforced option and before applying C_d.

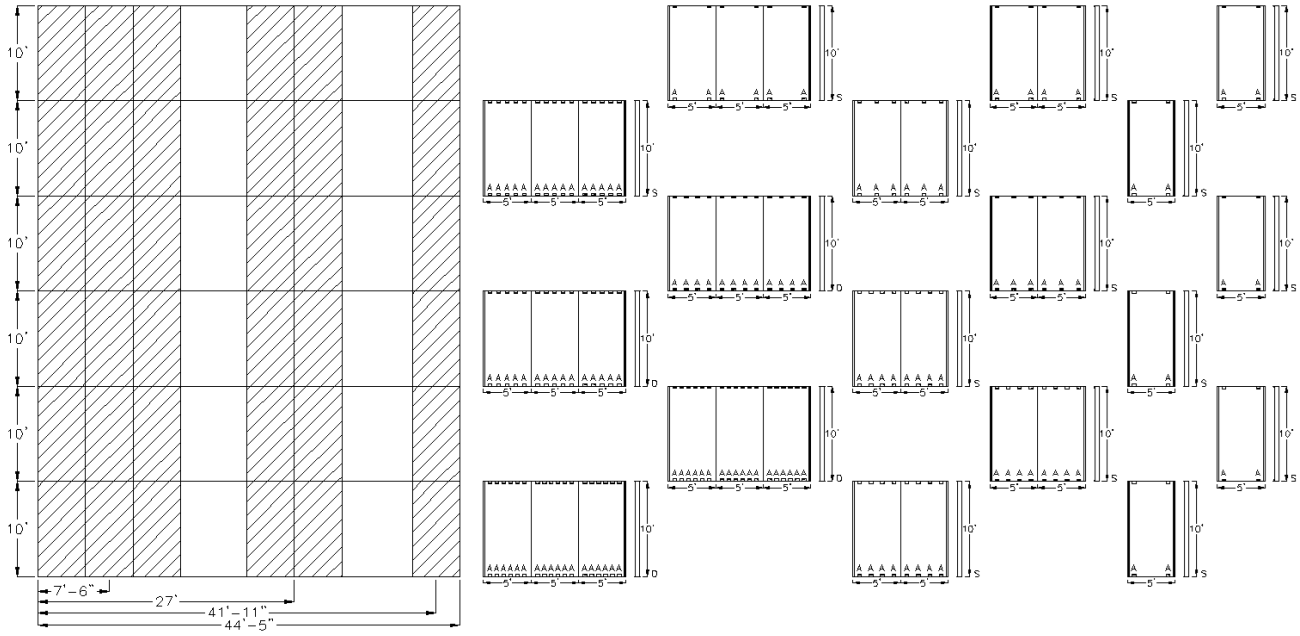


Figure B132. Archetype 89.

Table B123—Archetype 101 (8_B_6_2_ HR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.19	3.58	398.5	-	-	-
5	6	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.40	4.60	1104.6	-	-	-
4	6	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.48	5.29	1536.6	-	-	-
3	6	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/4	0.44	7.87	2092.0	-	-	-
2	6	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/2	0.50	7.94	2365.2	-	-	-
1	6	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/4	0.52	7.97	2482.9	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.19	2.27	380.0	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.40	2.29	824.5	-	-	-
4	4	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.48	3.33	1449.9	-	-	-
3	4	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.44	4.07	1623.5	-	-	-
2	4	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/4	0.50	4.06	1817.2	-	-	-
1	4	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/2	0.52	4.14	1935.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.19	0.94	314.4	31.26	11.35	2.75
5	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.40	1.02	732.0	39.08	28.47	1.37
4	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.48	1.03	893.8	57.31	42.02	1.36
3	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/4	0.44	1.11	881.5	72.94	52.02	1.40
2	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 3/8	0.50	1.10	980.9	72.94	58.55	1.25
1	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.52	1.09	1021.9	72.94	61.71	1.18

*Values shown are for reinforced option and before applying C_d.

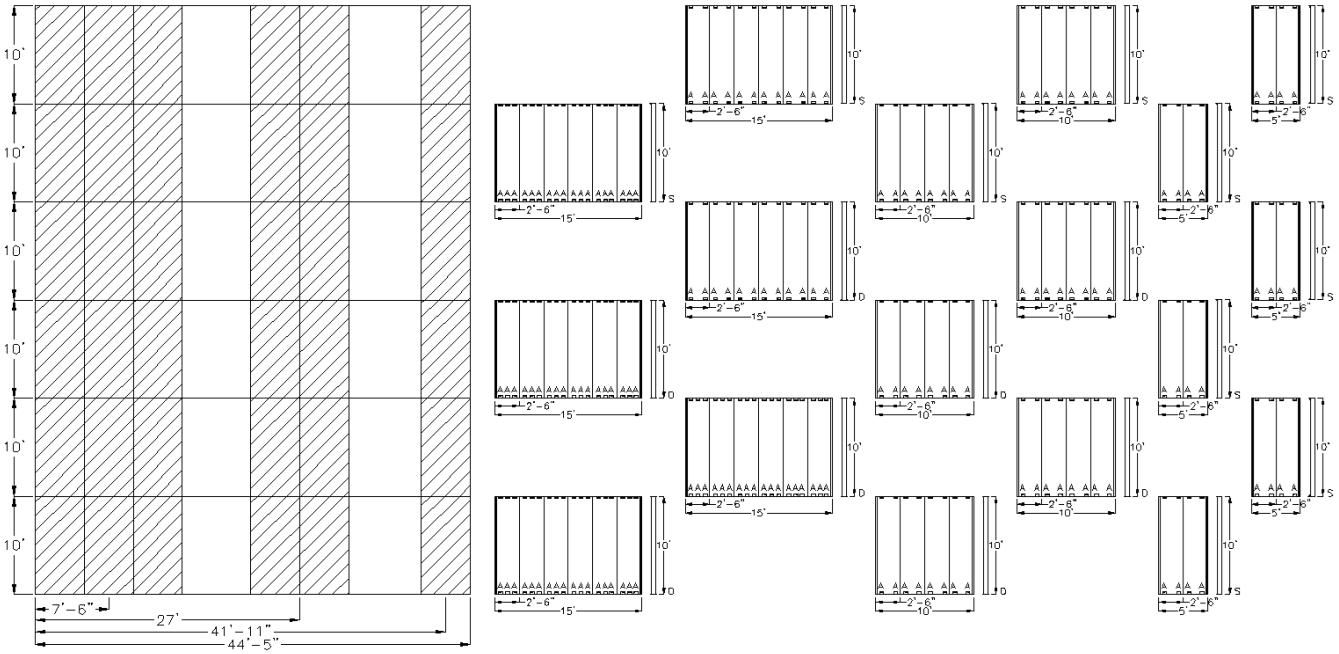


Figure B133. Archetype 101.

Table B124—Archetype 113 (8_B_6_2_ HR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.19	3.58	404.4	-	-	-
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.40	3.40	812.8	-	-	-
4	6	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.50	4.45	1331.4	-	-	-
3	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.46	6.63	1828.1	-	-	-
2	6	M	2.5	6.875	5	A3	2	D	2084.0	4	7/8	0.49	5.73	1680.9	-	-	-
1	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1	0.51	5.80	1786.7	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1/2	0.19	2.00	339.6	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1	0.40	2.38	854.2	-	-	-
4	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1	0.50	2.25	1006.8	-	-	-
3	4	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/8	0.46	2.47	1021.4	-	-	-
2	4	M	2.5	6.875	5	A3	2	D	2084.0	2	1 1/2	0.49	3.98	1752.6	-	-	-
1	4	M	2.5	6.875	5	A3	2	D	2084.0	2	1 3/4	0.51	4.00	1848.1	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.19	0.94	318.9	31.26	11.06	2.83
5	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.40	1.01	728.2	31.26	24.37	1.28
4	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.50	1.09	973.5	39.08	34.91	1.12
3	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.46	1.22	1010.8	46.89	42.69	1.10
2	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.49	1.14	1005.9	57.31	47.77	1.20
1	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.51	1.07	987.6	57.31	50.22	1.14

*Values shown are for reinforced option and before applying C_d.

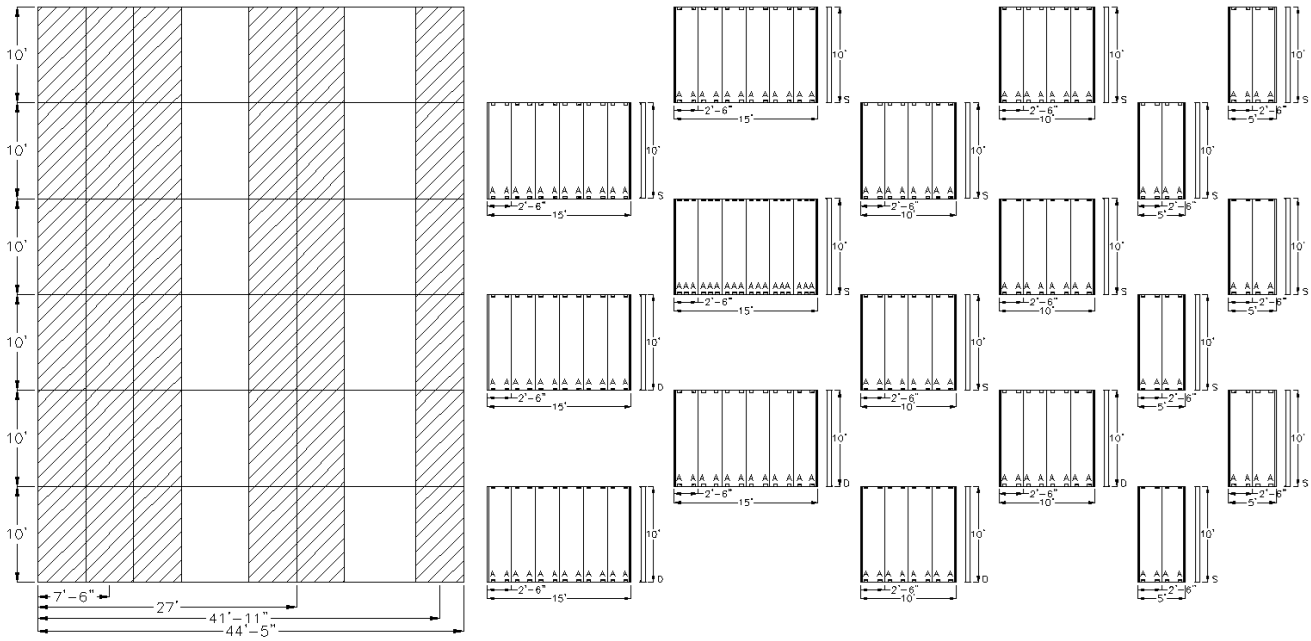


Figure B134. Archetype 113.

Table B125—Archetype 125 (8_B_6_2_ MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	3.58	371.0	-	-	-
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.31	3.58	656.5	-	-	-
4	6	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.32	5.30	1019.9	-	-	-
3	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.32	6.39	1228.7	-	-	-
2	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.34	6.48	1340.0	-	-	-
1	6	M	2.5	9.625	7	A3	2	D	2084.0	4	1 3/4	0.32	7.42	1442.9	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.17	2.32	360.4	-	-	-
5	2	M	5	4.125	3	A3	6	S	1563.0	4	5/8	0.31	5.43	1495.4	-	-	-
4	2	M	5	4.125	3	A3	5	D	2605.0	4	1	0.32	8.08	2332.3	-	-	-
3	2	M	5	6.875	5	A3	6	D	3126.0	4	1 1/4	0.32	10.34	2979.9	-	-	-
2	2	M	5	6.875	5	A3	7	D	3647.0	4	1 1/2	0.34	11.50	3567.9	-	-	-
1	2	M	5	9.625	7	A3	7	D	3647.0	4	1 3/4	0.32	12.48	3643.7	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	2	1/2	0.17	1.40	436.2	24.75	11.35	2.18
5	1	S	5	4.125	3	A3	3	S	781.5	2	5/8	0.31	1.33	734.7	35.17	28.47	1.24
4	1	S	5	4.125	3	A3	3	S	781.5	2	3/4	0.32	1.18	678.9	61.22	42.02	1.46
3	1	S	5	6.875	5	A3	3	S	781.5	2	7/8	0.32	1.32	758.6	66.43	52.02	1.28
2	1	S	5	6.875	5	A3	3	S	781.5	2	7/8	0.34	0.89	554.9	71.64	58.55	1.22
1	1	S	5	9.625	7	A3	4	S	1042.0	2	1	0.32	1.24	725.1	72.94	61.71	1.18

*Values shown are for reinforced option and before applying C_d.

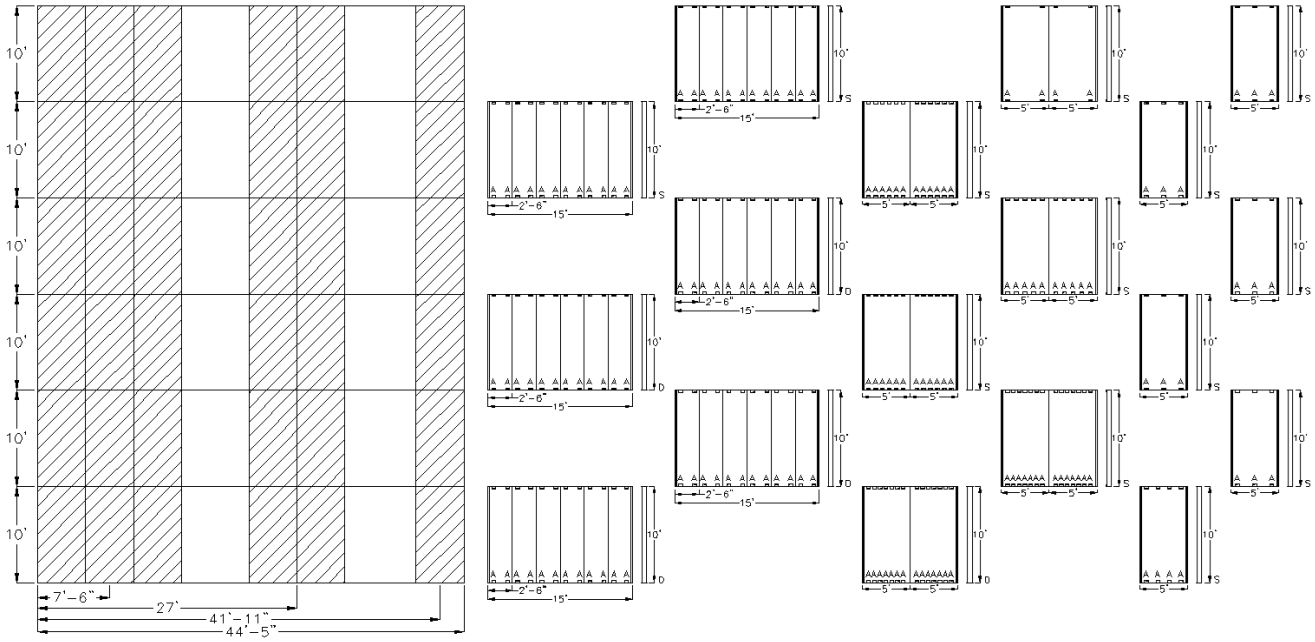


Figure B135. Archetype 125.

Table B126—Archetype 137 (8_B_6_2_ MR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	3.58	372.7	-	-	-
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.37	3.71	833.4	-	-	-
4	6	M	2.5	6.875	5	A3	2	D	2084.0	4	7/8	0.40	6.33	1499.3	-	-	-
3	6	M	2.5	9.625	7	A3	2	D	2084.0	4	1 1/8	0.38	7.14	1608.7	-	-	-
2	6	M	2.5	12.375	7	A3	2	D	2084.0	4	1 3/4	0.36	8.17	1774.0	-	-	-
1	6	M	2.5	12.375	7	A3	2	D	2084.0	4	1 3/4	0.40	8.06	1911.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	2.28	355.5	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.37	2.40	807.4	-	-	-
4	4	M	2.5	6.875	5	A3	2	S	1042.0	4	3/4	0.40	2.56	910.0	-	-	-
3	4	M	2.5	9.625	7	A3	2	D	2084.0	4	1	0.38	4.45	1505.1	-	-	-
2	4	M	2.5	12.375	7	A3	2	D	2084.0	4	1 3/4	0.36	5.34	1738.2	-	-	-
1	4	M	2.5	12.375	7	A3	2	D	2084.0	4	1 3/4	0.40	5.23	1858.3	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	4.125	3	A3	3	S	781.5	1	5/8	0.17	1.22	382.2	29.96	11.06	2.71
5	1	S	5	4.125	3	A3	3	S	781.5	1	3/4	0.37	1.13	759.7	29.96	24.37	1.23
4	1	S	5	6.875	5	A3	3	S	781.5	2	5/8	0.40	0.93	663.4	45.59	34.91	1.31
3	1	S	5	9.625	7	A3	3	S	781.5	2	3/4	0.38	1.04	701.6	56.01	42.69	1.31
2	1	S	5	12.375	7	A3	3	S	781.5	2	7/8	0.36	1.16	755.3	56.01	47.77	1.17
1	1	S	5	12.375	7	A3	3	S	781.5	2	7/8	0.40	0.83	593.0	56.01	50.22	1.12

*Values shown are for reinforced option and before applying C_d.

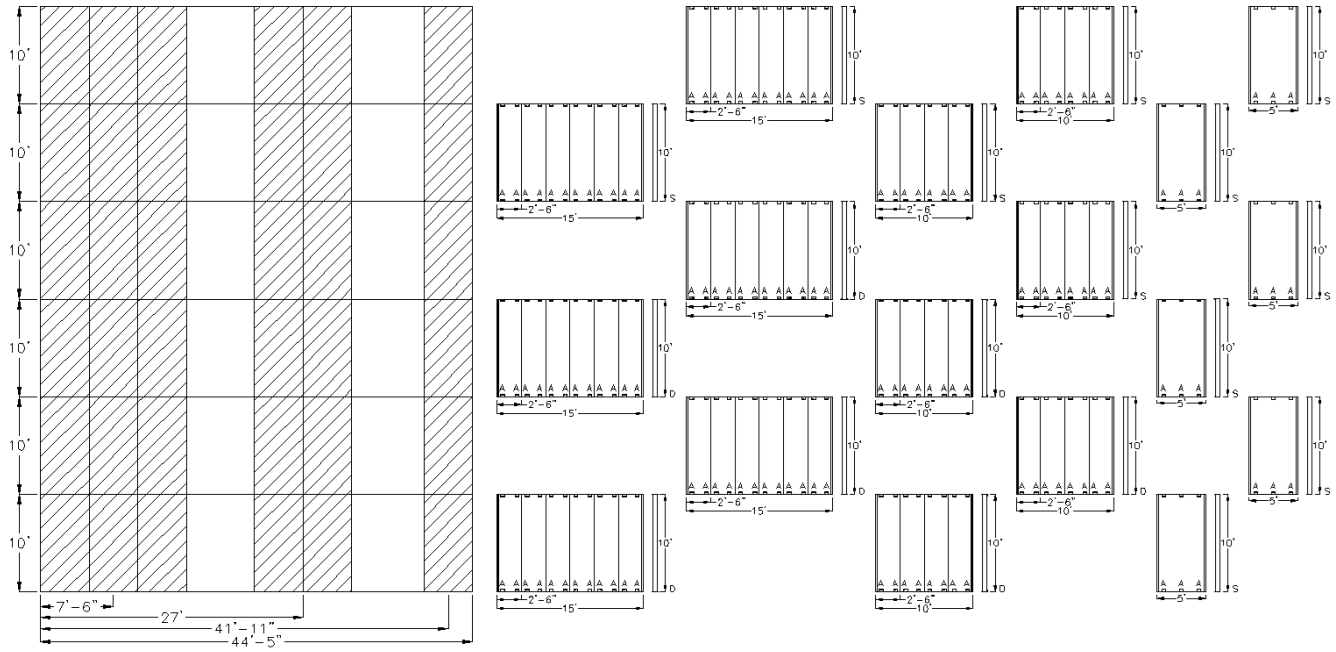


Figure B136. Archetype 137.

INPUT DATA

Total Height	h_n=	60.0	ft					
Total Weight	W=	2,442	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	4						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t (h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.6036						
	Ts=	0.6						
	Cs=	0.2485						
	k =	1.05	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	99,196						

DESIGN SUMMARY

Total base shear V = 606.93

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	294	21,839	0.220	133.6		
	10.00						133.6	
5		50.0	430	26,307	0.265	161.0		1,336
	10.00						294.6	
4		40.0	430	20,803	0.210	127.3		4,282
	10.00						421.9	
3		30.0	430	15,372	0.155	94.1		8,501
	10.00						515.9	
2		20.0	430	10,035	0.101	61.4		13,660
	10.00						577.3	
1		10.0	430	4,840	0.049	29.6		19,433
	10.00						606.9	
		0.0						25,502

Seismic base shear calculation, low gravity, Index Building 8, 6-story, R = 4

INPUT DATA

Total Height	h_n=	60.0	ft					
Total Weight	W=	2,970	k					
Seismic Design Category		Dmax						
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)					
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000		
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500		
	S_{DS} =	1.000	g ,					
	S_{D1} =	0.600	g					
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)					
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02						
The coefficient(ASCE Tab. 12.2.1)	R =	4						
	x =	0.75	, (ASCE Tab 12.8-2)					
	$T_a = C_t (h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)					
	Cu=	1.40						
	T=Cu*Ta=	0.6036						
	Ts=	0.6						
	Cs=	0.2485						
	k =	1.05	, (ASCE 12.8.3, pg 130)					
	$\Sigma w_x h^k$ =	118,211						

DESIGN SUMMARY

Total base shear V = 738.13

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	294	21,839	0.185	136.4		
	10.00						136.4	
5		50.0	535	32,773	0.277	204.6		1,364
	10.00						341.0	
4		40.0	535	25,917	0.219	161.8		4,774
	10.00						502.8	
3		30.0	535	19,150	0.162	119.6		9,802
	10.00						622.4	
2		20.0	535	12,501	0.106	78.1		16,026
	10.00						700.5	
1		10.0	535	6,030	0.051	37.7		23,031
	10.00						738.1	
		0.0						30,412

Seismic base shear calculation, high gravity, Index Building 8, 6-story, R = 4

Table B127—Tributary load calculation, Index Building 8, 6-story, N-S direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	1	565.0	0.097	133.62		136.36	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**16.29**		**16.63**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 5	1	565.0	0.097	294.58		341.01	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**35.92**		**41.58**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 4	1	565.0	0.097	421.86		502.84	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**51.44**		**61.31**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 3	1	565.0	0.097	515.91		622.42	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**62.91**		**75.89**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 2	1	565.0	0.097	577.31		700.48	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**70.39**		**85.41**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				
Story 1	1	565.0	0.097	606.93		738.13	
	2	957.6	0.164				
	3	**710.8**	**0.122**		**74.00**		**90.00**
	4	681.3	0.117				
	5	681.3	0.117				
	6	710.8	0.122				
	7	957.6	0.164				
	8	565.0	0.097				

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B128—Tributary load calculation, Index Building 8, 6-story, E-W direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	760.6	0.13	133.62		136.36	
	B	**1447.1**	**0.25**		**33.17**		**33.85**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 5	A	760.6	0.13	294.58		341.01	
	B	**1447.1**	**0.25**		**73.12**		**84.65**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 4	A	760.6	0.13	421.86		502.84	
	B	**1447.1**	**0.25**		**104.72**		**124.82**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 3	A	760.6	0.13	515.91		622.42	
	B	**1447.1**	**0.25**		**128.07**		**154.50**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 2	A	760.6	0.13	577.31		700.48	
	B	**1447.1**	**0.25**		**143.31**		**173.88**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				
Story 1	A	760.6	0.13	606.93		738.13	
	B	**1447.1**	**0.25**		**150.66**		**183.23**
	C	1414.2	0.24				
	D	1447.1	0.25				
	E	760.6	0.13				

* Seismic load for shear wall line B is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B129—Archetype 29 (8_3_6_1_ HR_HG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.13	3.66	277.1	15.63	4.16	3.76
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.31	3.67	693.0	15.63	10.40	1.50
4	6	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.46	3.67	1021.9	15.63	15.33	1.02
3	6	M	2.5	4.125	3	A3	3	S	1563.0	4	1	0.46	4.62	1264.9	23.45	18.97	1.24
2	6	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/8	0.44	5.44	1423.5	23.45	21.35	1.10
1	6	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/4	0.46	5.44	1500.1	23.45	22.50	1.04

*Values shown are for reinforced option and before applying C_d.

Table B130—Archetype 41 (8_3_6_1_ HR_LG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.12	3.66	271.5	15.63	4.07	3.84
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.27	3.66	598.6	15.63	8.98	1.74
4	6	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.39	3.66	857.3	15.63	12.86	1.22
3	6	M	2.5	4.125	3	A3	3	S	1563.0	4	1	0.38	4.61	1048.5	23.45	15.73	1.49
2	6	M	2.5	4.125	3	A3	3	S	1563.0	4	1 1/8	0.43	4.59	1173.2	23.45	17.60	1.33
1	6	M	2.5	6.875	5	A3	3	S	1563.0	4	1 3/8	0.37	5.51	1233.4	23.45	18.50	1.27

*Values shown are for reinforced option and before applying C_d.

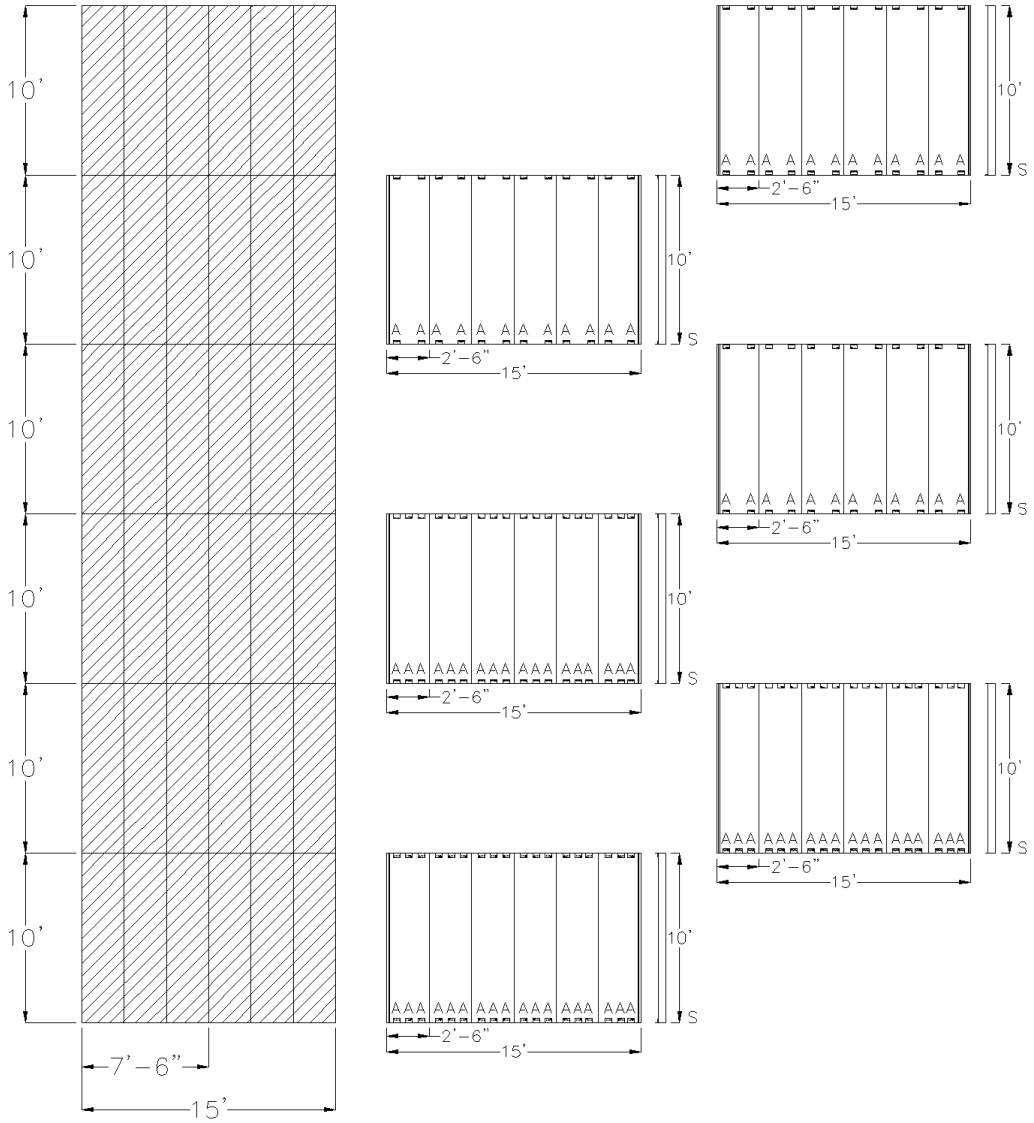


Figure B137. Archetype 29.

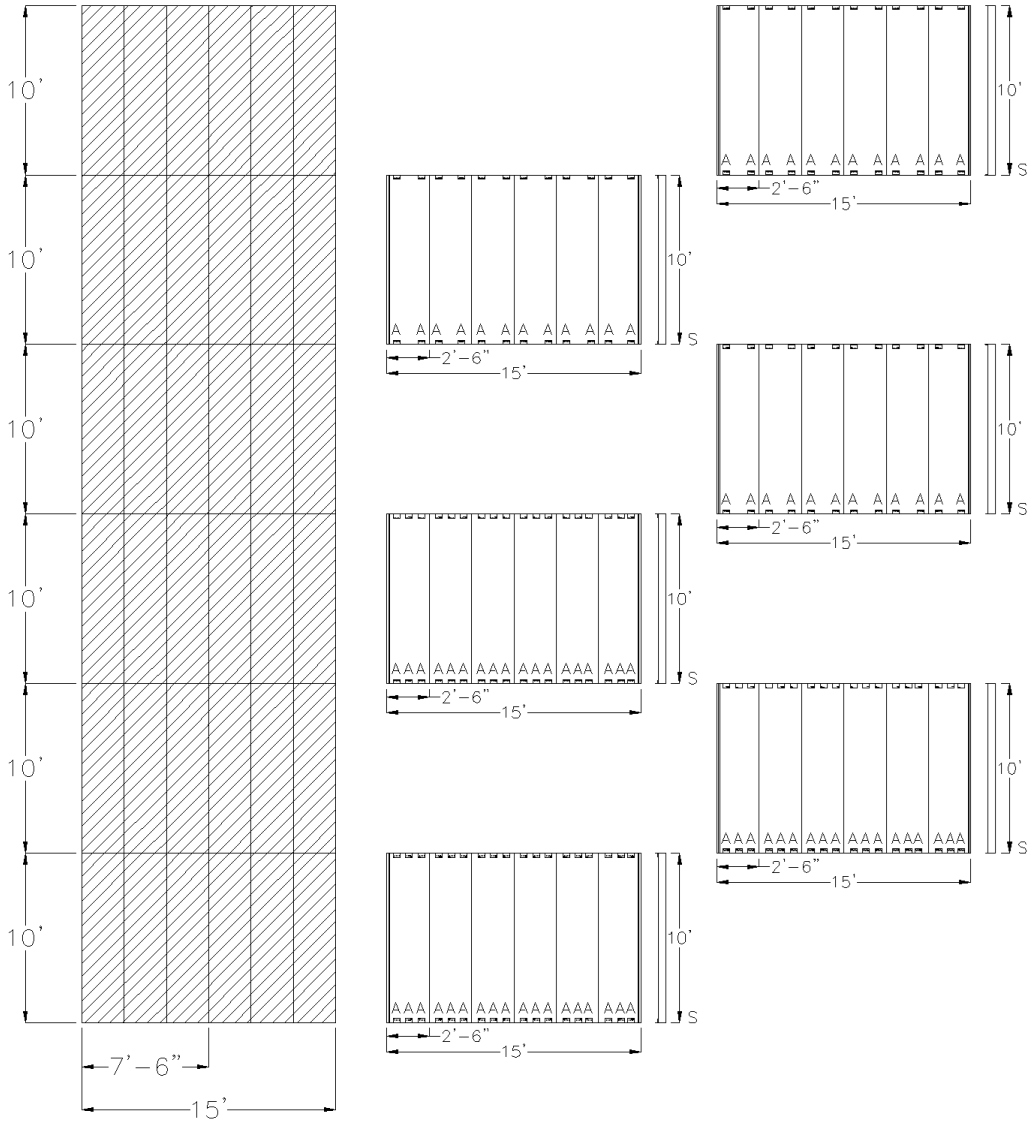


Figure B138. Archetype 41.

Table B131—Archetype 101 (8_B_6_2_ HR_HG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.14	3.59	296.6	-	-	-
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.34	3.61	733.7	-	-	-
4	6	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.40	5.33	1283.4	-	-	-
3	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/8	0.42	6.44	1629.1	-	-	-
2	6	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/8	0.42	7.86	1975.0	-	-	-
1	6	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/4	0.44	8.00	2091.8	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.14	2.29	283.3	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.34	2.30	702.2	-	-	-
4	4	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.40	2.28	822.9	-	-	-
3	4	M	2.5	6.875	5	A3	2	S	1042.0	4	7/8	0.42	2.60	985.9	-	-	-
2	4	M	2.5	6.875	5	A3	2	S	1042.0	4	1	0.42	2.58	973.3	-	-	-
1	4	M	2.5	6.875	5	A3	2	S	1042.0	4	1 1/8	0.44	2.59	1016.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.14	0.95	236.0	31.26	8.46	3.69
5	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.34	1.03	626.8	31.26	21.16	1.48
4	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.40	1.03	745.2	46.89	31.21	1.50
3	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/4	0.42	1.14	866.1	46.89	38.63	1.21
2	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 3/8	0.42	1.09	822.3	62.52	43.47	1.44
1	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.44	1.09	853.3	62.52	45.81	1.36

*Values shown are for reinforced option and before applying C_d.

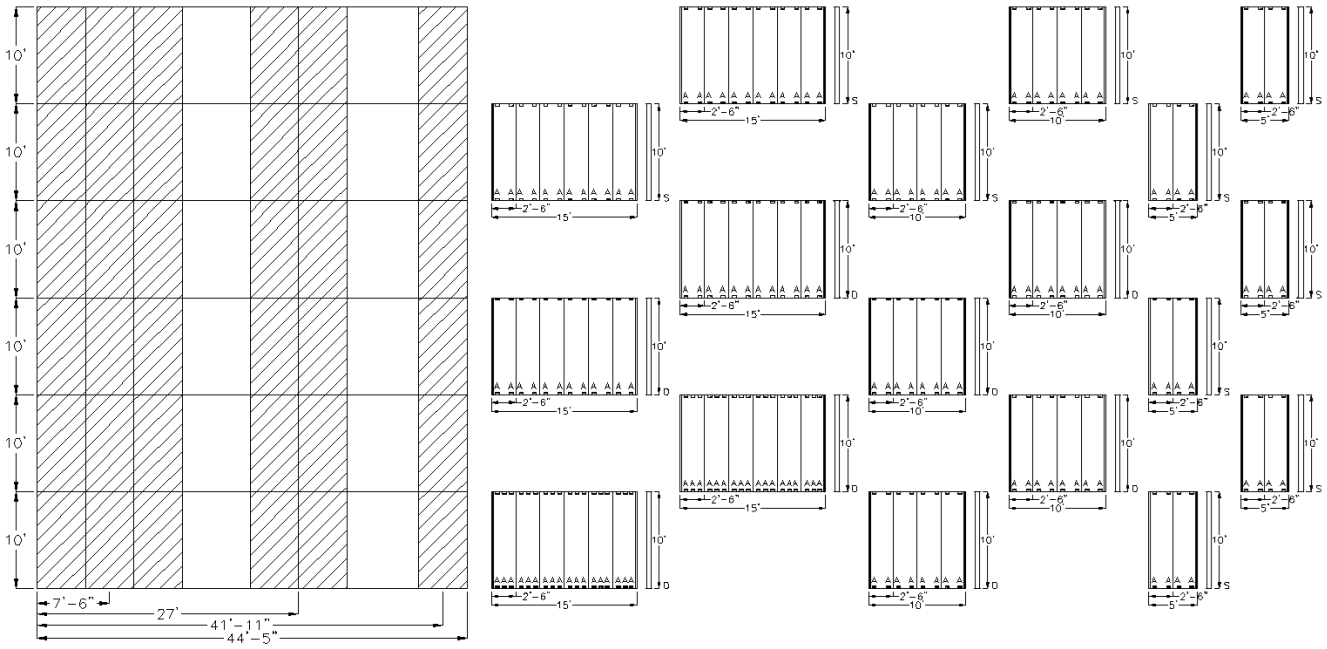


Figure B139. Archetype 101.

Table B132—Archetype 113 (8_B_6_2_ HR_LG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.14	3.59	293.8	-	-	-
5	6	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.30	3.41	616.8	-	-	-
4	6	M	2.5	4.125	3	A3	3	S	1563.0	4	5/8	0.38	4.25	973.1	-	-	-
3	6	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.47	4.30	1200.5	-	-	-
2	6	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.48	4.89	1413.3	-	-	-
1	6	M	2.5	6.875	5	A3	2	D	2084.0	4	1	0.40	5.86	1404.2	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.14	2.21	271.5	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.30	2.31	625.4	-	-	-
4	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.38	2.33	801.1	-	-	-
3	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.47	2.32	971.4	-	-	-
2	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/2	0.48	2.34	1012.9	-	-	-
1	4	M	2.5	6.875	5	A3	3	S	1563.0	2	1 3/4	0.40	3.44	1237.5	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.14	0.95	233.9	31.26	8.29	3.77
5	2	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.30	1.02	554.9	31.26	18.28	1.71
4	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.38	1.04	714.6	39.08	26.18	1.49
3	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.47	1.03	858.9	39.08	32.02	1.22
2	2	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/2	0.48	1.04	899.7	46.89	35.83	1.31
1	2	M	2.5	6.875	5	A3	2	S	1042.0	2	1 3/4	0.40	1.18	845.4	52.10	37.66	1.38

*Values shown are for reinforced option and before applying C_d.

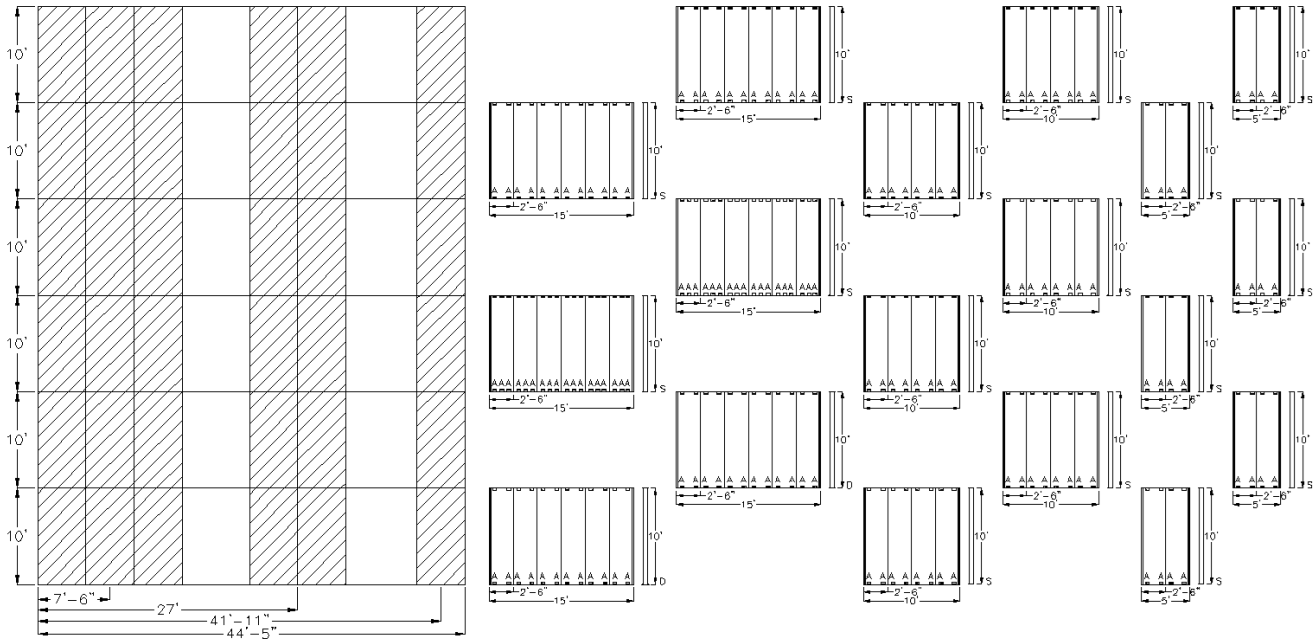


Figure B140. Archetype 113.

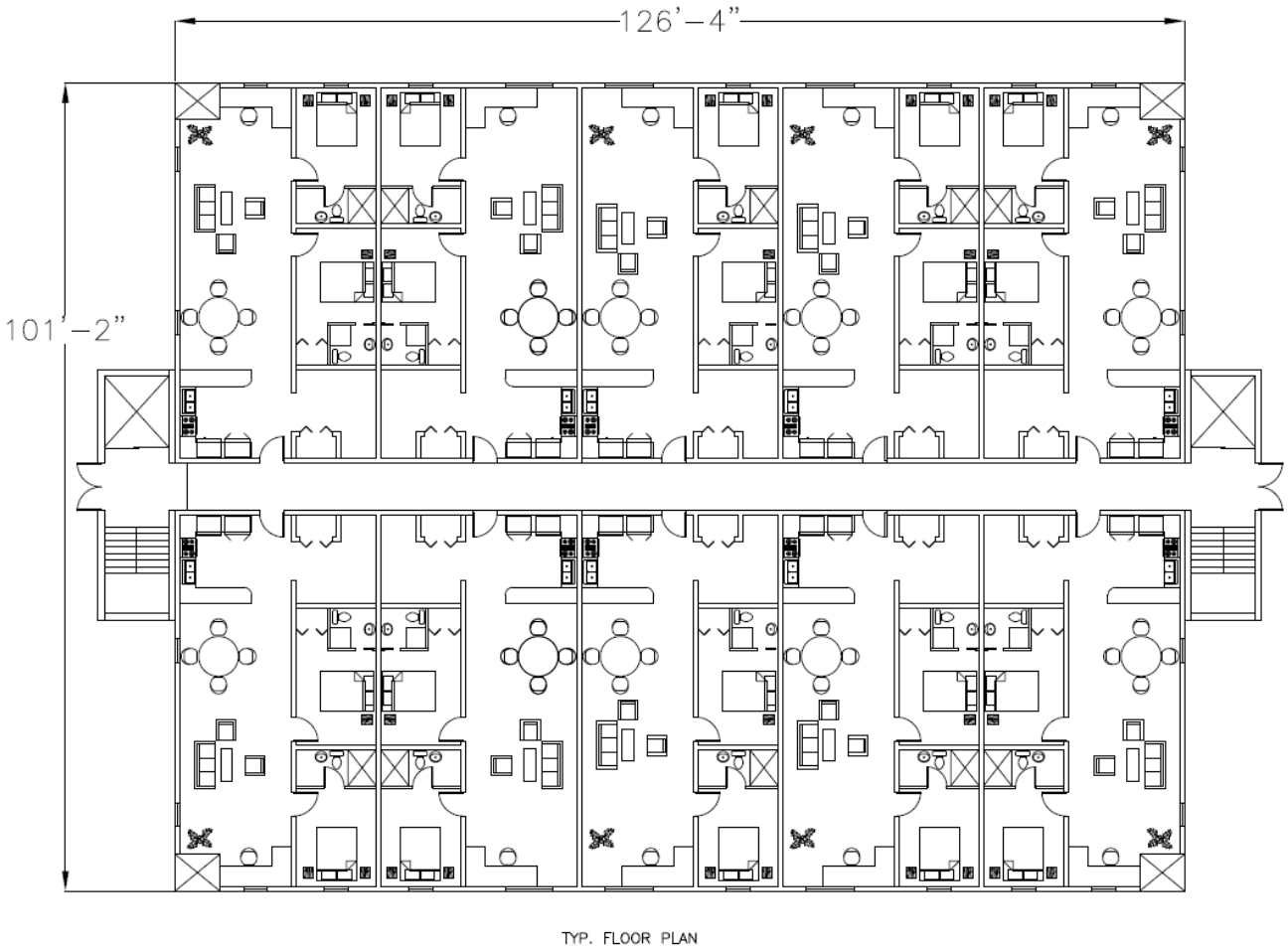


Figure B141. Index Building 9 floor plan.

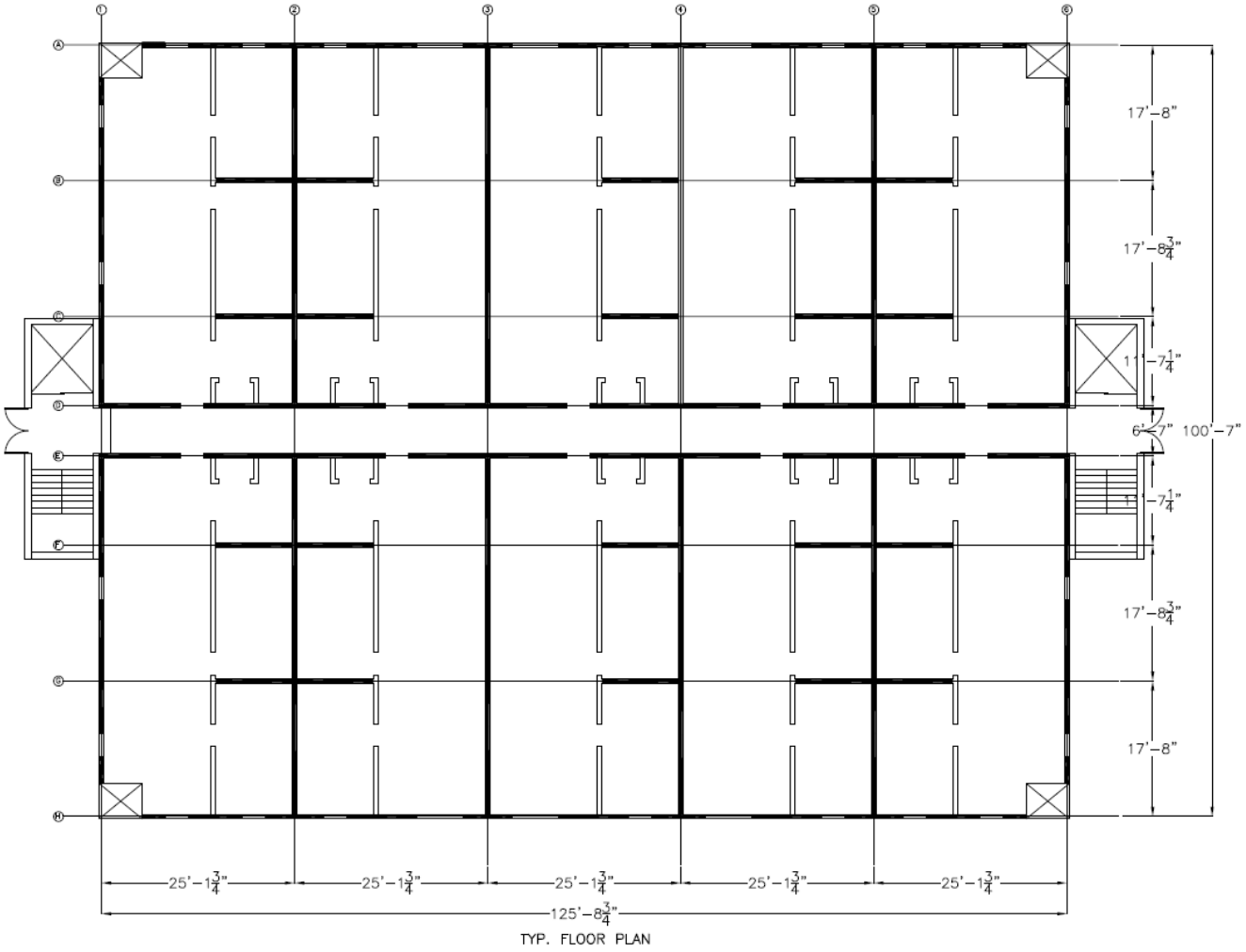


Figure B142. Index Building 9 assigned shear walls.

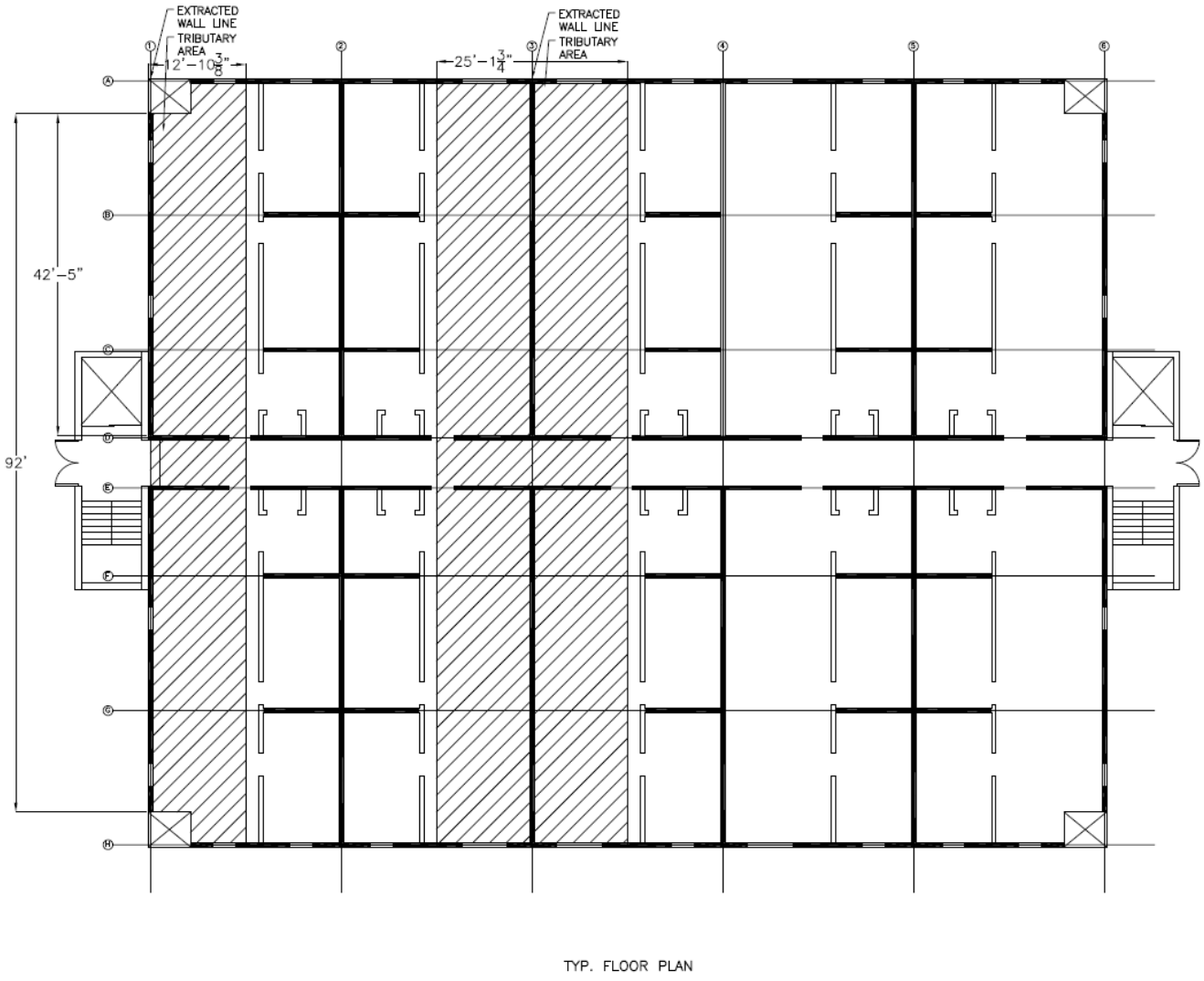


Figure B143. Index Building 9 tributary area.

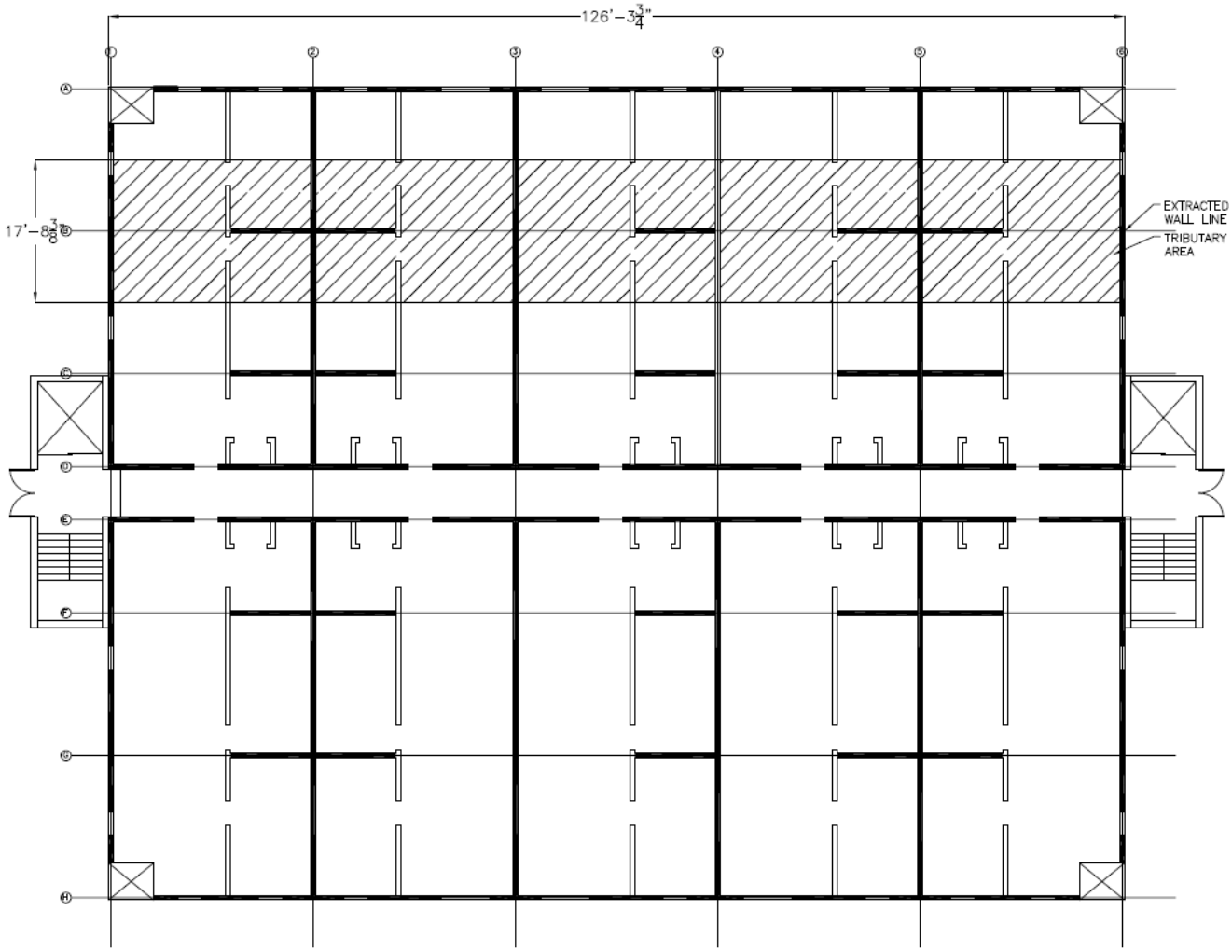


Figure B144. Index Building 9 tributary area.

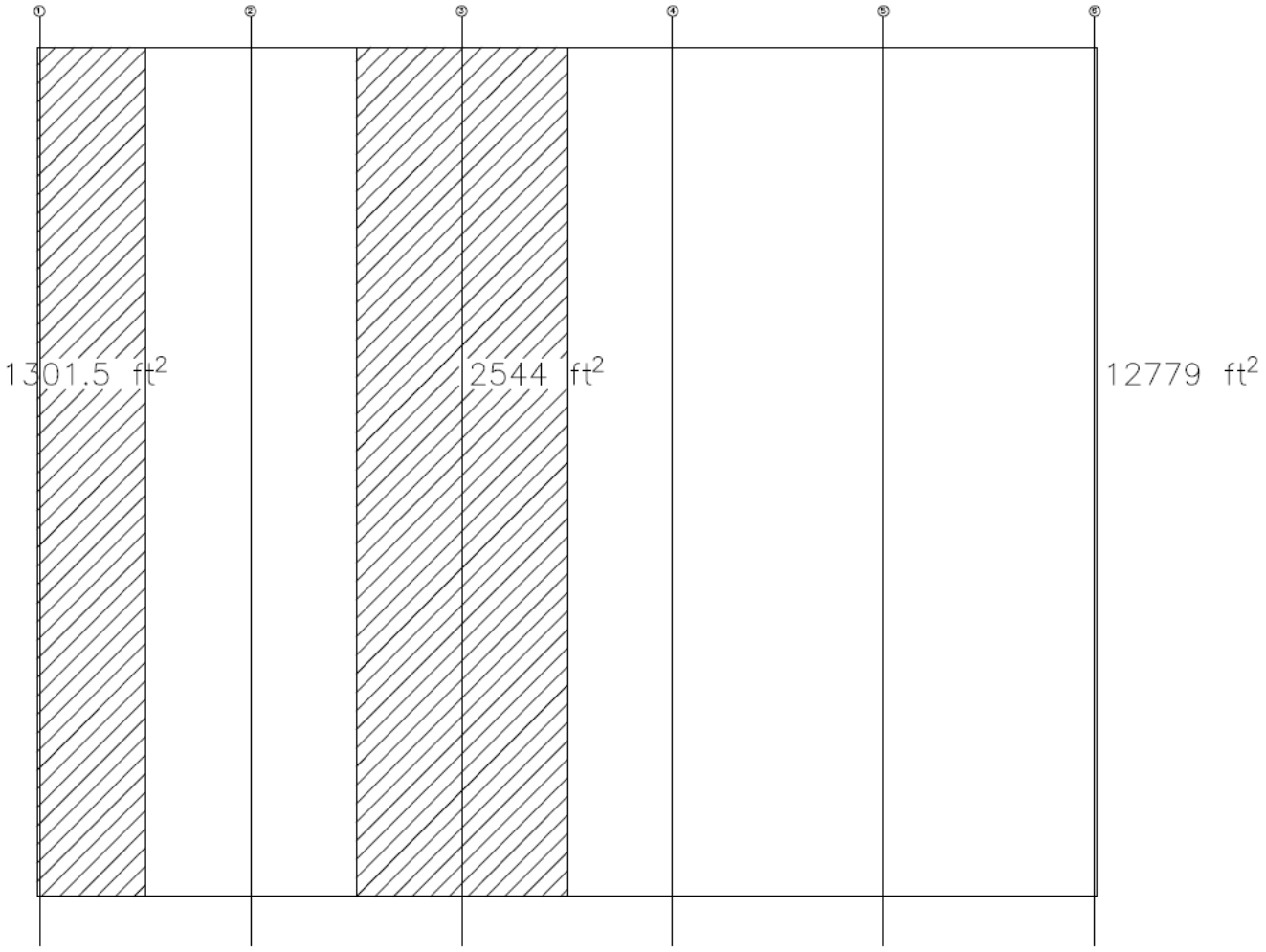


Figure B145. Index Building 9 tributary area.

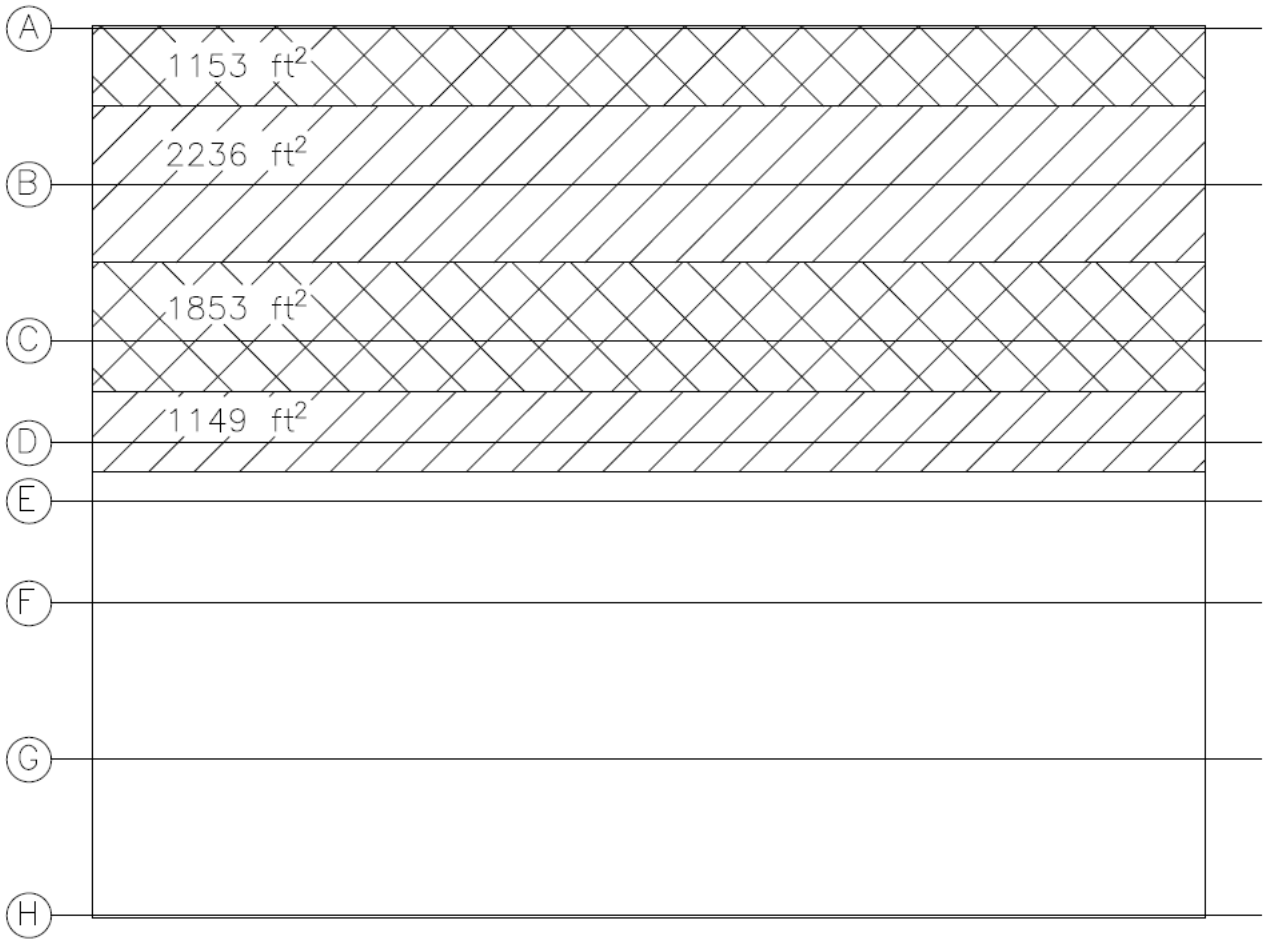


Figure B146. Index Building 9 tributary area.

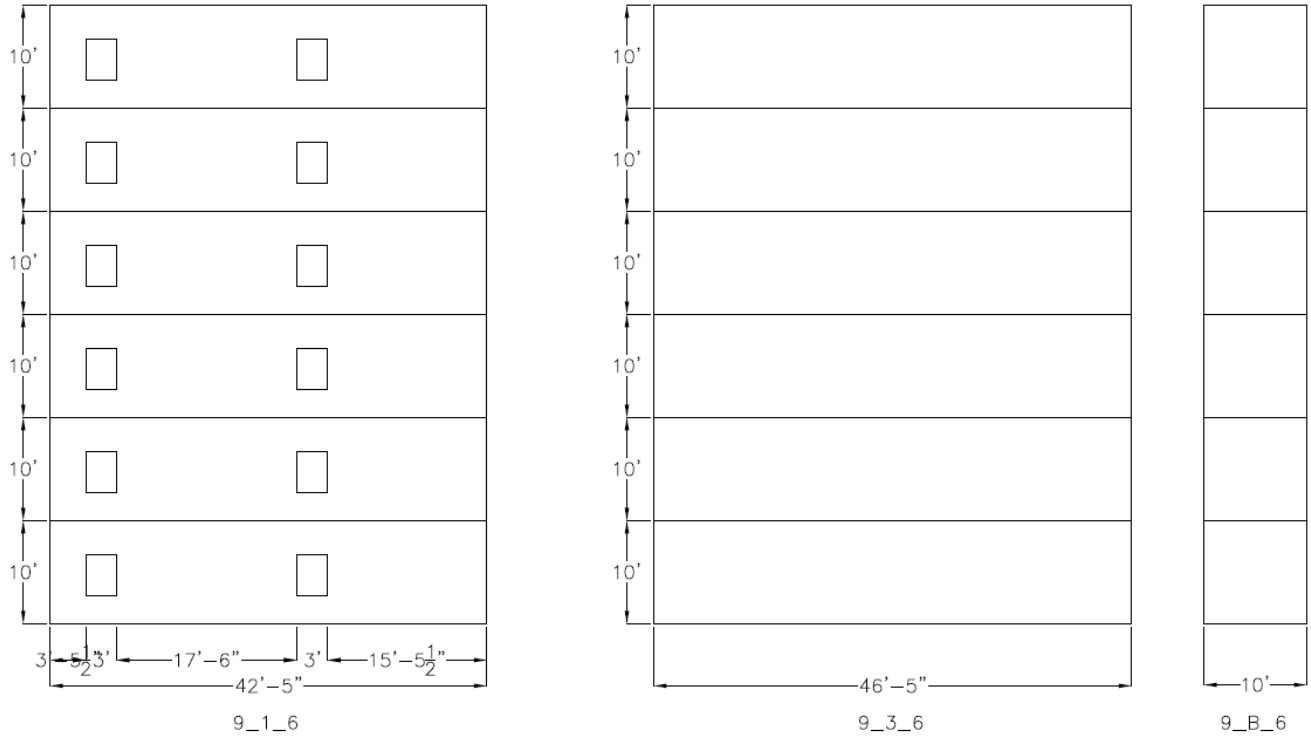


Figure B147. Index Building 9 extracted shear wall lines.

Table B133—Extracted archetypes, Index Building 9

Group No.		Basic Config.	Design Load Level: Gravity	Design Load Level: Seismic	Archetype description	Archetype No.
PG-2	2.5ft-20ft wall	Low aspect ratio panels	High	SDC D_{max}	9_B_6 _1_ LR_HG_ DX_LP	6
PG-6			Low		9_B_6 _1_ LR_LG_ DX_LP	18
PG-10		High aspect ratio panels	High		9_B_6 _1_HR_HG_ DX_LP	30
PG-14			Low		9_B_6 _1_ HR_LG_ DX_LP	42
PG-26	20ft-60ft wall	Low aspect ratio panels	High		9_3_6_2_ LR_HG_ DX_LP	78
PG-30			Low		9_3_6_2_ LR_LG_ DX_LP	90
PG-34		High aspect ratio panels	High		9_3_6 _2_ HR_HG_ DX_LP	102
PG-38			Low		9_3_6 _2_ HR_LG_ DX_LP	114
PG-42		Mixed aspect ratio	High		9_3_6_2_ MR_HG_ DX_LP	126
PG-46			Low		9_3_6_2_ MR_LG_ DX_LP	138

Table B134—Seismic weight detailed calculation, Index Building 9, 6-story

								Low gravity				High gravity			
Level	Story	*h* (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)	W_{floor} (lb)	$W_{extWalls}$ (lb)	$W_{IntWalls}$ (lb)	ΣW_{Level} (lb)
Roof			12779					373998.7			596927	373998.7			596927
	6	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
5			12779					399130.8			844988	630590.4			1076448
	5	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
4			12779					399130.8			844988	630590.4			1076448
	4	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
3			12779					399130.8			844988	630590.4			1076448
	3	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
2			12779					399130.8			844988	630590.4			1076448
	2	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
1			12779					399130.8			844988	630590.4			1076448
	1	10		453	4042.00	1287.00	11937		136300	309558			136300	309558	
Ground			0												

Table B135—Seismic weight summary for Index Building 9, 6-story

Level	*h* (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		597	597
	10		
5		845	1076.4
	10		
4		845	1076.4
	10		
3		845	1076.4
	10		
2		845	1076.4
	10		
1		845	1076.4
	10		
Ground			

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	4,822	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	196,435					

DESIGN SUMMARY

Total base shear

V = 1,597.63

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	597	44,278	0.225	360.1		
	10.00						360.1	
5		50.0	845	51,744	0.263	420.8		3,601
	10.00						781.0	
4		40.0	845	40,919	0.208	332.8		11,411
	10.00						1,113.8	
3		30.0	845	30,235	0.154	245.9		22,548
	10.00						1,359.7	
2		20.0	845	19,738	0.100	160.5		36,145
	10.00						1,520.2	
1		10.0	845	9,521	0.048	77.4		51,347
	10.00						1,597.6	
		0.0						67,323

Seismic base shear calculation, low gravity, Index Building 9, 6-story, R = 3

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	5,979	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	3					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	238,103					

DESIGN SUMMARY

Total base shear

V = 1,980.98

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	597	44,278	0.186	368.4		
	10.00						368.4	
5		50.0	1,076	65,914	0.277	548.4		3,684
	10.00						916.8	
4		40.0	1,076	52,125	0.219	433.7		12,852
	10.00						1,350.4	
3		30.0	1,076	38,515	0.162	320.4		26,356
	10.00						1,670.9	
2		20.0	1,076	25,143	0.106	209.2		43,065
	10.00						1,880.1	
1		10.0	1,076	12,128	0.051	100.9		61,866
	10.00						1,981.0	
		0.0						81,676

Seismic base shear calculation, high gravity, Index Building 9, 6-story, R = 3

Table B136—Tributary load calculation, Index Building 9, 6-story, N-S direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load	Story shear (kip)	Cumulative Shear Load
Story 6	1	1301.5	0.102	360.12		368.39	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**71.69**		**73.34**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 5	1	1301.5	0.102	780.96		916.78	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**155.47**		**182.51**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 4	1	1301.5	0.102	1113.76		1350.45	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**221.72**		**268.84**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 3	1	1301.5	0.102	1359.67		1670.89	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**270.68**		**332.64**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 2	1	1301.5	0.102	1520.20		1880.08	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**302.64**		**374.28**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 1	1	1301.5	0.102	1597.63		1980.98	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**318.05**		**394.37**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B137—Tributary load calculation, Index Building 9, 6-story, E-W direction, R = 3

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	1153.00	0.09	360.12		368.39	
	B	**2236.00**	**0.17**		**63.00**		**64.44**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 5	A	1153.00	0.09	780.96		916.78	
	B	**2236.00**	**0.17**		**136.62**		**160.38**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 4	A	1153.00	0.09	1113.76		1350.45	
	B	**2236.00**	**0.17**		**194.83**		**236.24**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 3	A	1153.00	0.09	1359.67		1670.89	
	B	**2236.00**	**0.17**		**237.85**		**292.29**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 2	A	1153.00	0.09	1520.20		1880.08	
	B	**2236.00**	**0.17**		**265.93**		**328.89**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 1	A	1153.00	0.09	1597.63		1980.98	
	B	**2236.00**	**0.17**		**279.48**		**346.54**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				

* Seismic load for shear wall line B is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B138—Archetype 6 (9_B_6 _1_ LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	3	S	781.5	4	3/8	0.25	2.91	644.5	7.82	6.45	1.21
5	2	M	5	4.125	3	A3	4	D	2084.0	4	3/4	0.26	6.72	1603.4	20.84	16.03	1.30
4	2	M	5	4.125	3	A3	5	D	2605.0	4	1	0.33	7.90	2361.7	26.05	23.62	1.10
3	2	M	5	4.125	3	A3	6	D	3126.0	4	1 3/8	0.35	9.21	2922.1	31.26	29.22	1.07
2	2	M	5	4.125	3	A3	7	D	3647.0	4	1 3/4	0.36	10.15	3287.9	36.47	32.88	1.11
1	2	M	5	6.875	5	A3	7	D	3647.0	4	2	0.32	12.10	3464.3	36.47	34.64	1.05

*Values shown are for reinforced option and before applying C_d.

Table B139—Archetype 18 (9_B_6 _1_ LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	4.125	3	A3	3	S	781.5	4	3/8	0.24	2.92	630.0	7.82	6.30	1.24
5	2	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.26	5.73	1366.2	15.63	13.66	1.14
4	2	M	5	4.125	3	A3	4	D	2084.0	4	1	0.31	7.05	1948.3	20.84	19.48	1.07
3	2	M	5	4.125	3	A3	5	D	2605.0	4	1 1/4	0.32	8.14	2378.5	26.05	23.79	1.10
2	2	M	5	4.125	3	A3	6	D	3126.0	4	1 1/2	0.33	8.99	2659.3	31.26	26.59	1.18
1	2	M	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.29	10.55	2794.8	31.26	27.95	1.12

*Values shown are for reinforced option and before applying C_d.

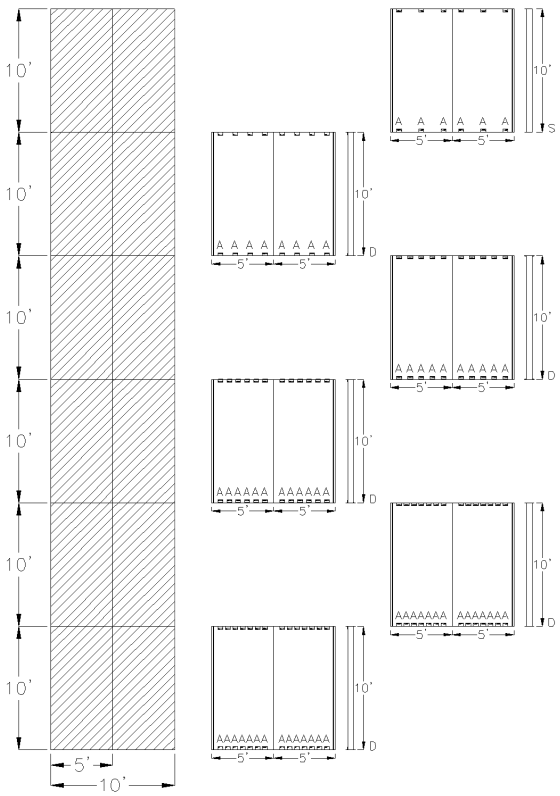


Figure B148. Archetype 6.

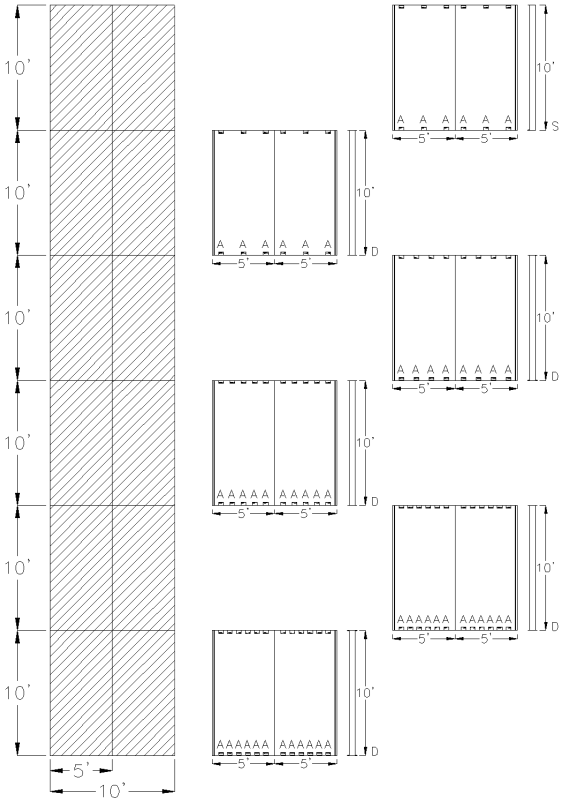


Figure B149. Archetype 18.

Table B140—Archetype 30 (9_B_6 _1_HR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.32	2.26	644.5	10.42	6.45	1.62
5	4	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.52	3.43	1603.4	20.84	16.03	1.30
4	4	M	2.5	4.125	3	A3	3	D	3126.0	4	1 1/4	0.64	4.09	2361.7	31.26	23.62	1.32
3	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/2	0.62	5.20	2922.1	31.26	29.22	1.07
2	4	M	2.5	6.875	5	A3	4	D	4168.0	4	1 3/4	0.63	5.79	3287.9	41.68	32.88	1.27
1	4	M	2.5	6.875	5	A3	4	D	4168.0	4	2	0.66	5.80	3464.3	41.68	34.64	1.20

*Values shown are for reinforced option and before applying C_d.

Table B141—Archetype 42 (9_B_6 _1_ HR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.31	2.26	630.0	10.42	6.30	1.65
5	4	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.44	3.42	1366.2	20.84	13.66	1.53
4	4	M	2.5	4.125	3	A3	2	D	2084.0	4	1 1/8	0.63	3.46	1948.3	20.84	19.48	1.07
3	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/8	0.52	5.08	2378.5	31.26	23.79	1.31
2	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/2	0.59	5.05	2659.3	31.26	26.59	1.18
1	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/4	0.61	5.08	2794.8	31.26	27.95	1.12

*Values shown are for reinforced option and before applying C_d.

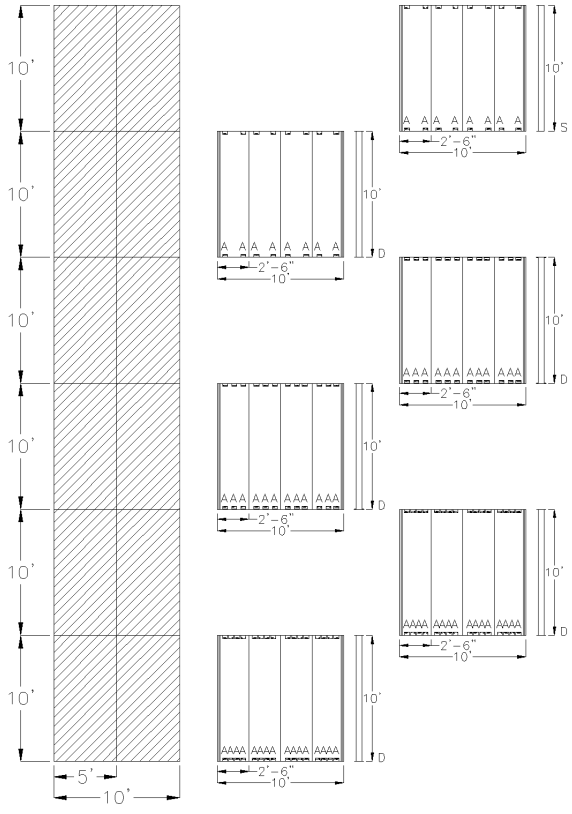


Figure B150. Archetype 30.

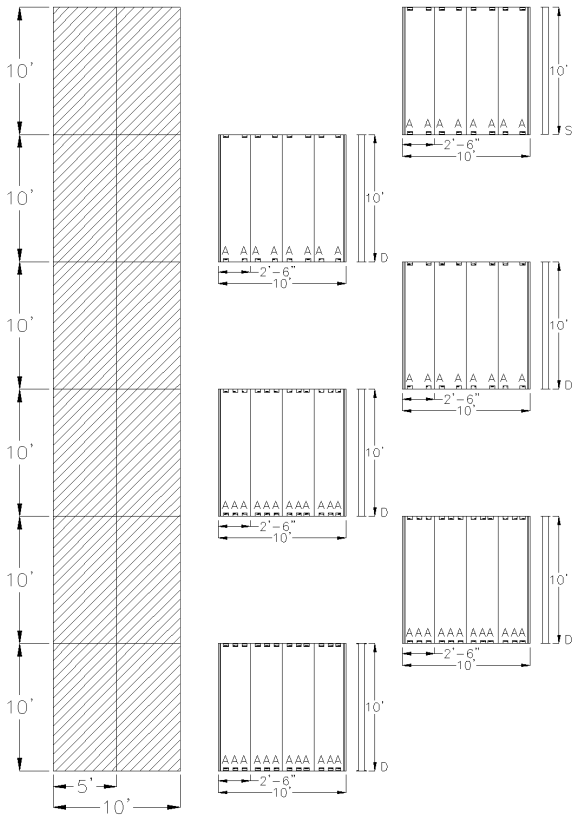


Figure B151. Archetype 42.

Table B142—Archetype 78 (9_3_6_2_ LR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	9	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.16	12.96	407.4	23.45	18.33	1.28
5	9	M	5	4.125	3	A3	4	S	1042.0	4	1/2	0.22	22.73	1013.9	46.89	45.63	1.03
4	9	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.24	31.25	1493.6	70.34	67.21	1.05
3	9	M	5	4.125	3	A3	4	D	2084.0	4	1	0.24	38.33	1848.0	93.78	83.16	1.13
2	9	M	5	6.875	5	A3	4	D	2084.0	4	11/4	0.24	43.39	2079.3	93.78	93.57	1.00
1	9	M	5	6.875	5	A3	5	D	2605.0	4	13/8	0.22	50.32	2190.9	117.23	98.59	1.19

*Values shown are for reinforced option and before applying C_d.

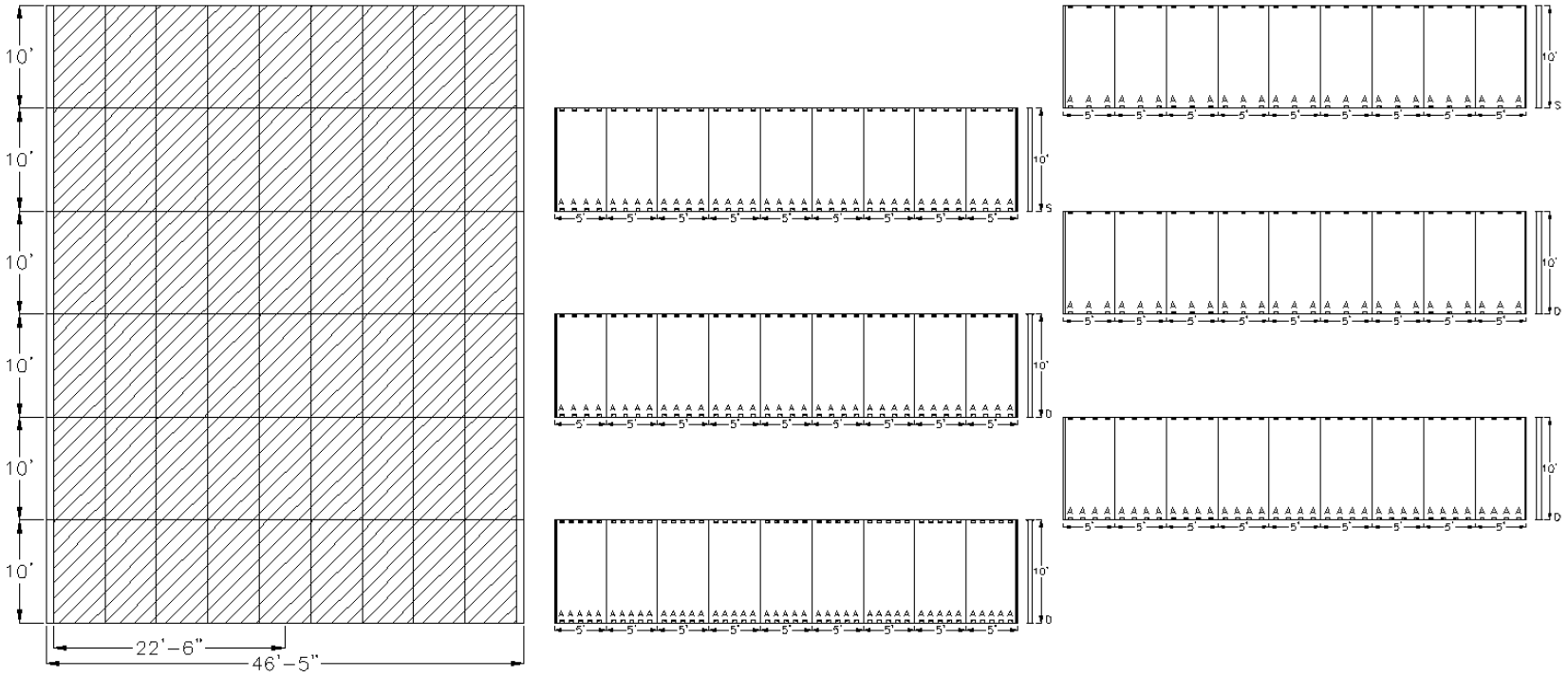


Figure B152. Archetype 78.

Table B143—Archetype 90 (9_3_6_2_ LR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.18	5.41	448.1	-	-	-
5	4	M	5	4.125	3	A3	4	S	1042.0	4	5/8	0.22	9.70	971.7	-	-	-
4	4	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.24	12.69	1385.8	-	-	-
3	4	M	5	4.125	3	A3	4	D	2084.0	4	1	0.24	15.65	1691.7	-	-	-
2	4	M	5	6.875	5	A3	4	D	2084.0	4	1 1/4	0.24	17.88	1891.5	-	-	-
1	4	M	5	6.875	5	A3	4	D	2084.0	4	1 3/8	0.25	17.73	1987.8	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.18	5.41	448.1	20.84	17.92	1.16
5	4	M	5	4.125	3	A3	4	S	1042.0	4	5/8	0.22	9.70	971.7	41.68	38.87	1.07
4	4	M	5	4.125	3	A3	3	D	1563.0	4	3/4	0.24	12.69	1385.8	62.52	55.43	1.13
3	4	M	5	4.125	3	A3	4	D	2084.0	4	1	0.24	15.65	1691.7	83.36	67.67	1.23
2	4	M	5	6.875	5	A3	4	D	2084.0	4	1 1/4	0.24	17.88	1891.5	83.36	75.66	1.10
1	4	M	5	6.875	5	A3	4	D	2084.0	4	1 3/8	0.25	17.73	1987.8	83.36	79.51	1.05

*Values shown are for reinforced option and before applying C_d.

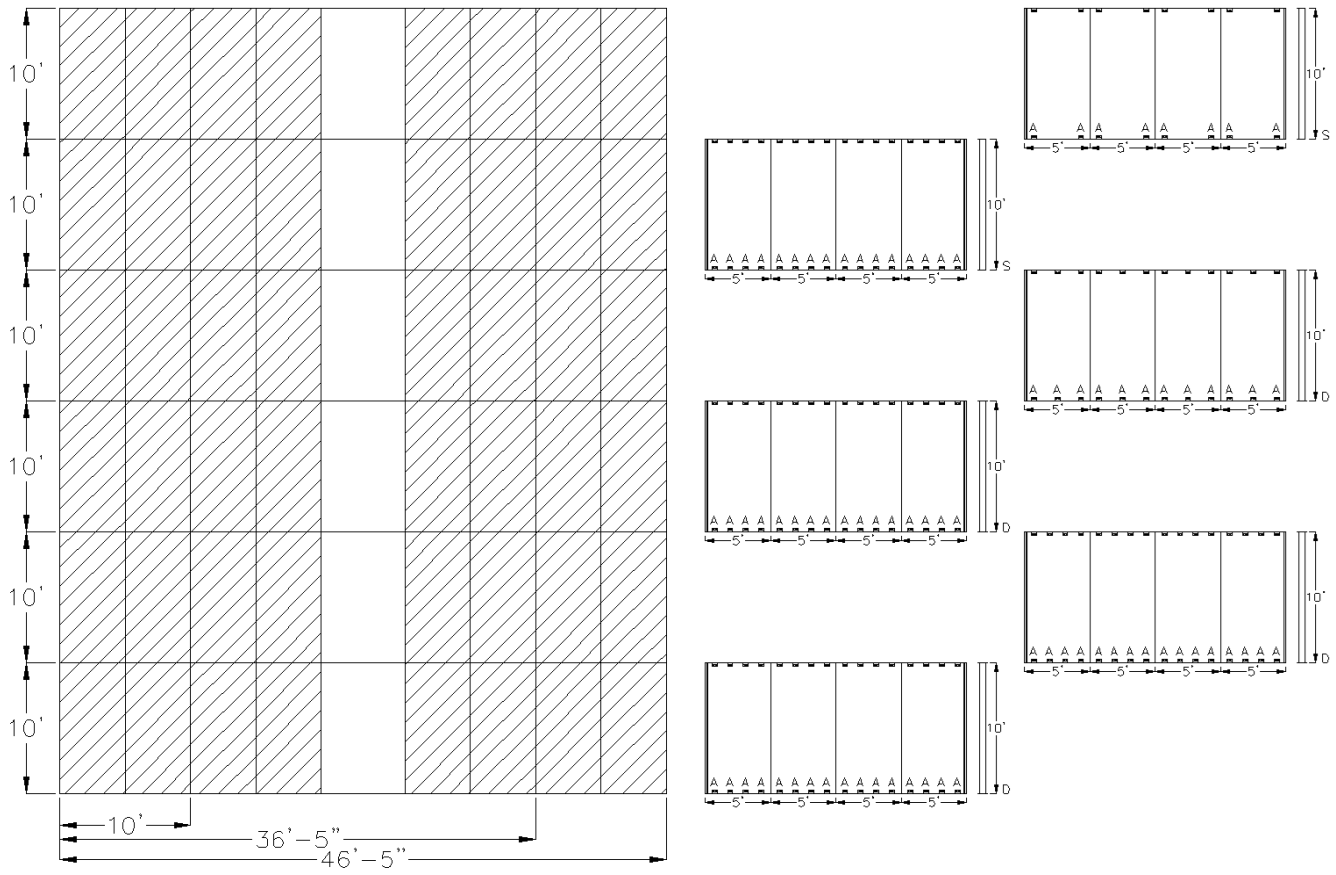


Figure B153. Archetype 90.

Table B144—Archetype 102 (9_3_6 _2_ HR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.18	11.53	407.4	46.89	18.33	2.56
5	18	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.44	11.56	1013.9	46.89	45.63	1.03
4	18	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.45	16.74	1493.6	93.78	67.21	1.40
3	18	M	2.5	4.125	3	A3	2	D	2084.0	4	11/4	0.55	16.79	1848.0	93.78	83.16	1.13
2	18	M	2.5	6.875	5	A3	2	D	2084.0	4	13/8	0.51	20.57	2079.3	93.78	93.57	1.00
1	18	M	2.5	6.875	5	A3	3	D	3126.0	4	13/4	0.44	25.14	2190.9	140.67	98.59	1.43

*Values shown are for reinforced option and before applying C_d.

Table B145—Archetype 114 (9_3_6 _2_ HR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.17	11.53	398.3	46.89	17.92	2.62
5	18	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.37	11.54	863.7	46.89	38.87	1.21
4	18	M	2.5	4.125	3	A3	2	D	2084.0	4	7/8	0.37	16.60	1231.8	93.78	55.43	1.69
3	18	M	2.5	4.125	3	A3	2	D	2084.0	4	1 1/8	0.45	16.69	1503.8	93.78	67.67	1.39
2	18	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.41	20.56	1681.3	93.78	75.66	1.24
1	18	M	2.5	6.875	5	A3	2	D	2084.0	4	1 1/2	0.43	20.54	1767.0	93.78	79.51	1.18

*Values shown are for reinforced option and before applying C_d.

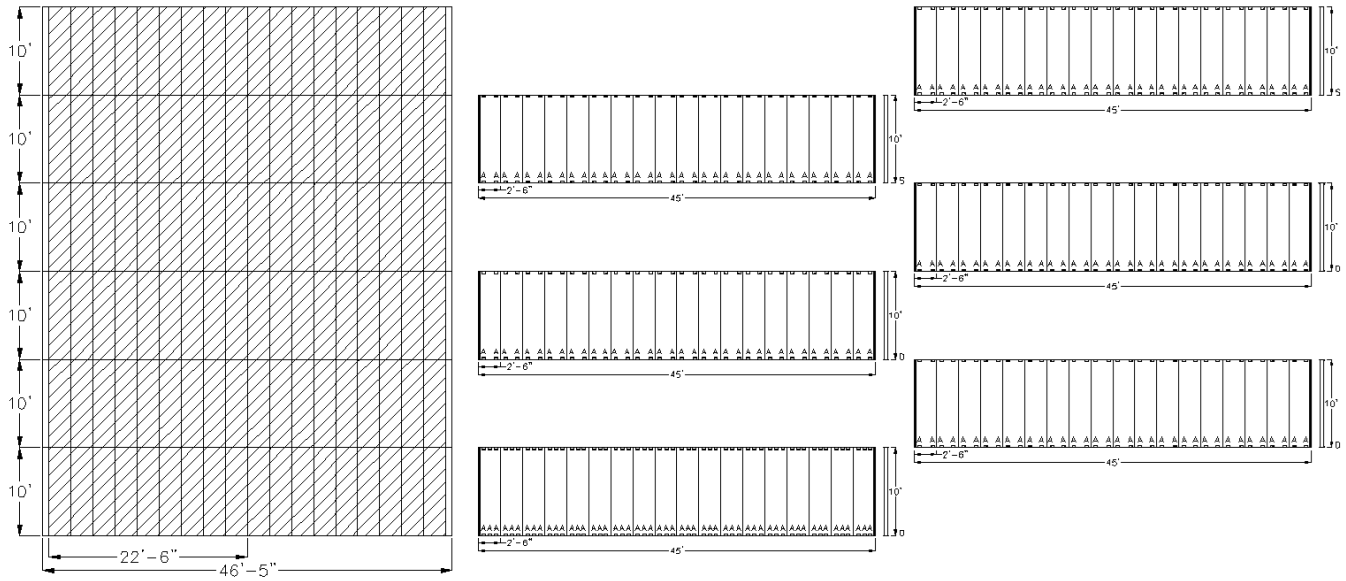


Figure B154. Archetype 102.

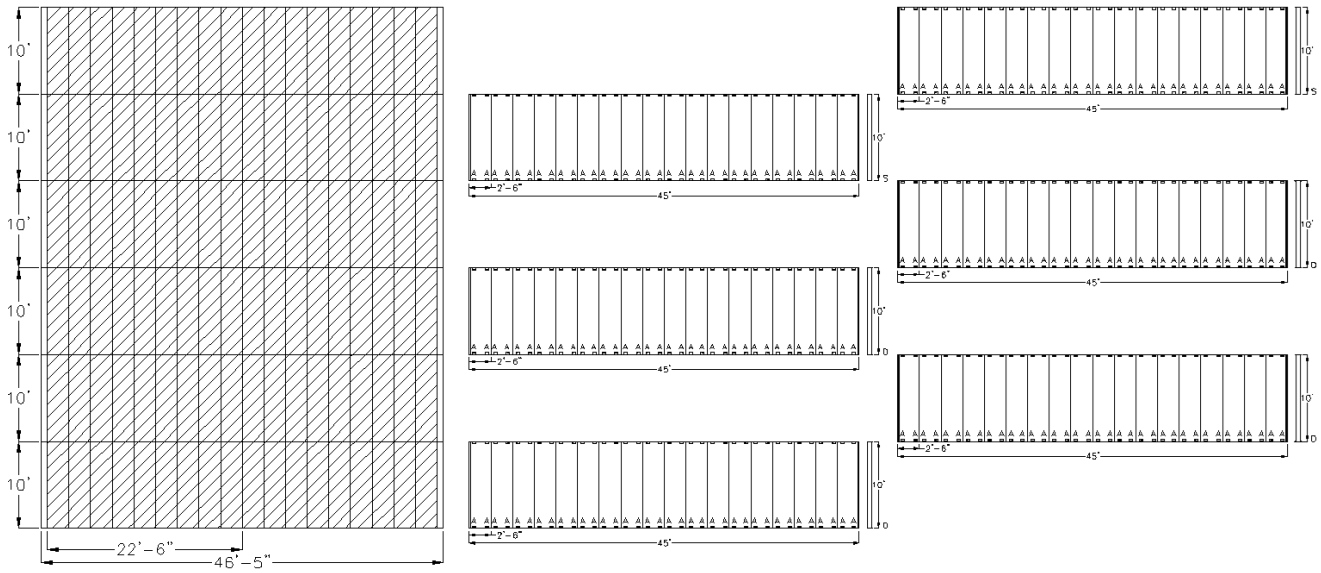


Figure B155. Archetype 114.

Table B146—Archetype 126 (9_3_6_2_ MR_HG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.20	3.90	474.7	-	-	-
5	3	M	5	4.125	3	A3	3	D	1563.0	4	5/8	0.25	9.17	1351.4	-	-	-
4	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.30	11.36	2030.5	-	-	-
3	3	M	5	4.125	3	A3	5	D	2605.0	4	1 1/8	0.32	13.13	2546.1	-	-	-
2	3	M	5	6.875	5	A3	6	D	3126.0	4	1 3/8	0.29	16.88	2895.7	-	-	-
1	3	M	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.29	17.54	3059.5	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.20	2.24	409.4	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.25	2.30	508.4	-	-	-
4	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.30	2.35	629.4	-	-	-
3	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.32	2.33	677.5	-	-	-
2	4	M	2.5	6.875	5	A3	2	S	1042.0	2	1 3/8	0.29	2.60	669.8	-	-	-
1	4	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.29	2.60	680.5	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	2	S	521.0	4	3/8	0.20	3.90	474.7	26.05	18.33	1.42
5	3	M	5	4.125	3	A3	3	D	1563.0	4	5/8	0.25	9.17	1351.4	57.31	45.63	1.26
4	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.30	11.36	2030.5	72.94	67.21	1.09
3	3	M	5	4.125	3	A3	5	D	2605.0	4	1 1/8	0.32	13.13	2546.1	88.57	83.16	1.07
2	3	M	5	6.875	5	A3	6	D	3126.0	4	1 3/8	0.29	16.88	2895.7	104.20	93.57	1.11
1	3	M	5	6.875	5	A3	6	D	3126.0	4	1 3/4	0.29	17.54	3059.5	104.20	98.59	1.06

*Values shown are for reinforced option and before applying C_d.

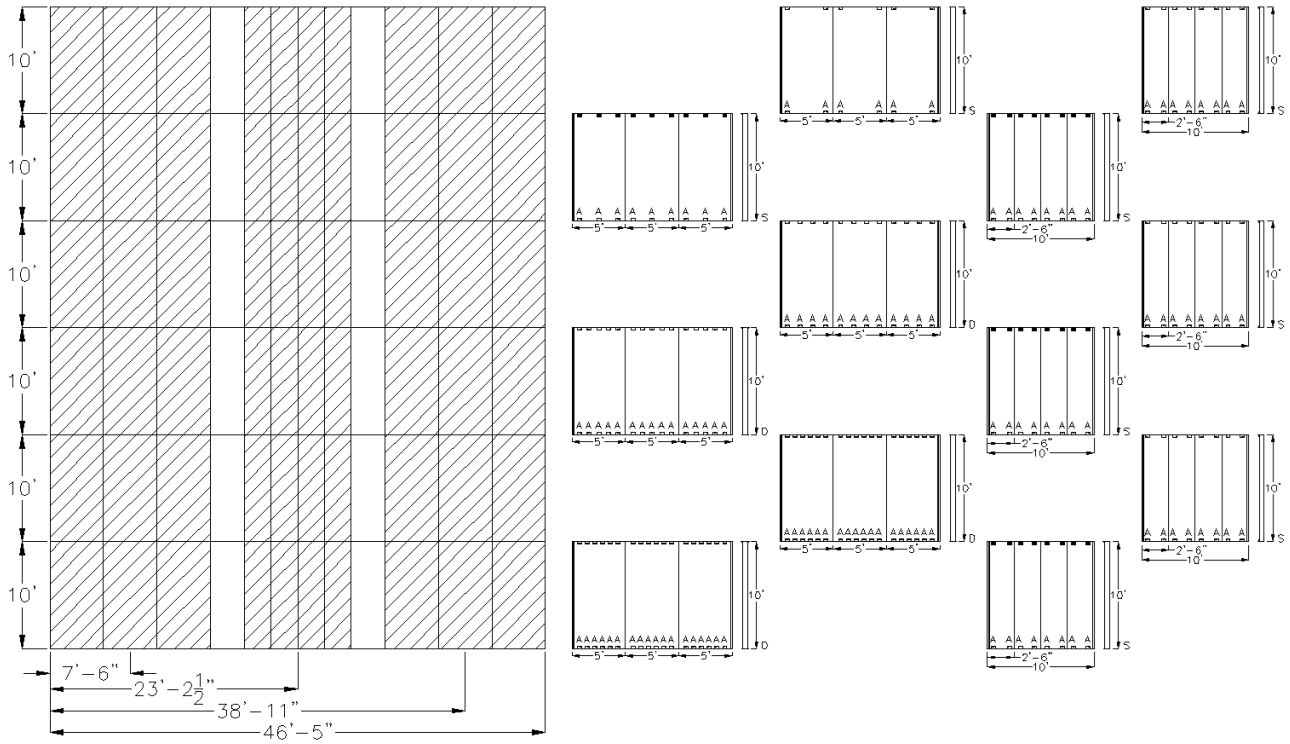


Figure B156. Archetype 126.

Table B147—Archetype 138 (9_3_6_2_ MR_LG_ DX_LP) design, R = 3

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	4	S	1042.0	4	1/2	0.12	7.00	513.2	-	-	-
5	3	M	5	4.125	3	A3	5	S	1302.5	4	5/8	0.23	8.07	1130.7	-	-	-
4	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.25	11.27	1673.1	-	-	-
3	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.32	10.54	2028.6	-	-	-
2	3	M	5	4.125	3	A3	5	D	2605.0	4	1	0.33	11.70	2294.7	-	-	-
1	3	M	5	6.875	5	A3	5	D	2605.0	4	1 1/8	0.31	13.14	2409.3	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	2	5/8	0.12	2.30	252.5	-	-	-
5	4	M	2.5	4.125	3	A3	2	S	1042.0	2	7/8	0.23	2.35	494.7	-	-	-
4	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/8	0.25	2.35	523.7	-	-	-
3	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 1/4	0.32	2.36	681.1	-	-	-
2	4	M	2.5	4.125	3	A3	2	S	1042.0	2	1 3/8	0.33	2.32	681.8	-	-	-
1	4	M	2.5	6.875	5	A3	2	S	1042.0	2	1 1/2	0.31	2.63	723.4	-	-	-

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	4.125	3	A3	4	S	1042.0	4	1/2	0.12	7.00	513.2	41.68	17.92	2.33
5	3	M	5	4.125	3	A3	5	S	1302.5	4	5/8	0.23	8.07	1130.7	49.50	38.87	1.27
4	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.25	11.27	1673.1	72.94	55.43	1.32
3	3	M	5	4.125	3	A3	4	D	2084.0	4	7/8	0.32	10.54	2028.6	72.94	67.67	1.08
2	3	M	5	4.125	3	A3	5	D	2605.0	4	1	0.33	11.70	2294.7	88.57	75.66	1.17
1	3	M	5	6.875	5	A3	5	D	2605.0	4	1 1/8	0.31	13.14	2409.3	88.57	79.51	1.11

*Values shown are for reinforced option and before applying C_d.

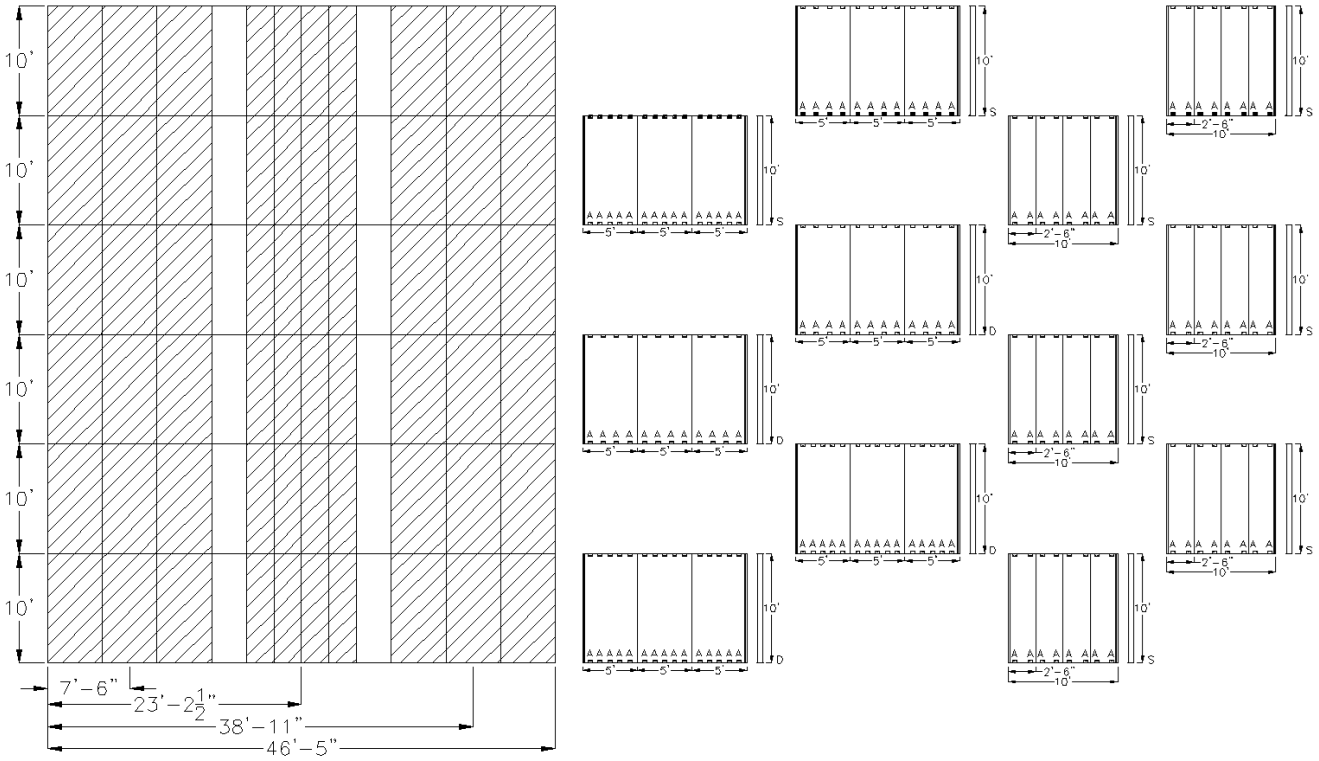


Figure B157. Archetype 138.

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	4,822	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.2485					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	196,435					

DESIGN SUMMARY

Total base shear V = 1,198.22

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	597	44,278	0.225	270.1		
	10.00						270.1	
5		50.0	845	51,744	0.263	315.6		2,701
	10.00						585.7	
4		40.0	845	40,919	0.208	249.6		8,558
	10.00						835.3	
3		30.0	845	30,235	0.154	184.4		16,911
	10.00						1,019.8	
2		20.0	845	19,738	0.100	120.4		27,109
	10.00						1,140.1	
1		10.0	845	9,521	0.048	58.1		38,510
	10.00						1,198.2	
		0.0						50,493

Seismic base shear calculation, low gravity, Index Building 9, 6-story, R = 4

INPUT DATA

Total Height	h_n=	60.0	ft				
Total Weight	W=	5,979	k				
Seismic Design Category		Dmax					
Importance factor (ASCE 11.5.1)	I =	1	(IBC Tab.1604.5)				
	S_S =	1.500	%g , S_{ms} =	1.500	g , F_a =	1.000	
	S_1 =	0.600	%g , S_{m1} =	0.900	g , F_v =	1.500	
	S_{DS} =	1.000	g ,				
	S_{D1} =	0.600	g				
Site class (A, B, C, D, E, F)		D	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02					
The coefficient(ASCE Tab. 12.2.1)	R =	4					
	x =	0.75	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	1.40					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.2485					
	k =	1.05	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	238,103					

DESIGN SUMMARY

Total base shear V = 1,485.73

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	597	44,278	0.186	276.3		
	10.00						276.3	
5		50.0	1,076	65,914	0.277	411.3		2,763
	10.00						687.6	
4		40.0	1,076	52,125	0.219	325.3		9,639
	10.00						1,012.8	
3		30.0	1,076	38,515	0.162	240.3		19,767
	10.00						1,253.2	
2		20.0	1,076	25,143	0.106	156.9		32,299
	10.00						1,410.1	
1		10.0	1,076	12,128	0.051	75.7		46,399
	10.00						1,485.7	
		0.0						61,257

Seismic base shear calculation, high gravity, Index Building 9, 6-story, R = 4

Table B148—Tributary load calculation, Index Building 9, 6-story, N-S direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	1	1301.5	0.102	270.13		276.29	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**53.78**		**55.00**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 5	1	1301.5	0.102	585.75		687.58	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**116.61**		**136.88**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 4	1	1301.5	0.102	835.35		1012.84	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**166.30**		**201.63**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 3	1	1301.5	0.102	1019.78		1253.17	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**203.01**		**249.48**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 2	1	1301.5	0.102	1140.17		1410.06	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**226.98**		**280.71**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				
Story 1	1	1301.5	0.102	1198.25		1485.73	
	2	2544.0	0.199				
	3	**2544.0**	**0.199**		**238.54**		**295.77**
	4	2544.0	0.199				
	5	2544.0	0.199				
	8	1301.5	0.102				

* Seismic load for shear wall line 3 is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B149—Tributary load calculation, Index Building 9, 6-story, E-W direction, R = 4

				Low gravity		High gravity	
	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Story shear (kip)	Cumulative Shear Load (kip)*	Story shear (kip)	Cumulative Shear Load (kip)*
Story 6	A	1153.00	0.09	270.13		276.29	
	B	**2236.00**	**0.17**		**47.25**		**48.33**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 5	A	1153.00	0.09	585.75		687.58	
	B	**2236.00**	**0.17**		**102.47**		**120.28**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 4	A	1153.00	0.09	835.35		1012.84	
	B	**2236.00**	**0.17**		**146.13**		**177.18**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 3	A	1153.00	0.09	1019.78		1253.17	
	B	**2236.00**	**0.17**		**178.39**		**219.22**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 2	A	1153.00	0.09	1140.17		1410.06	
	B	**2236.00**	**0.17**		**199.45**		**246.67**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				
Story 1	A	1153.00	0.09	1198.25		1485.73	
	B	**2236.00**	**0.17**		**209.61**		**259.90**
	C	1853.00	0.14				
	D	1149.00	0.09				
	E	1149.00	0.09				
	F	1853.00	0.14				
	G	2236.00	0.17				
	H	1153.00	0.09				

* Seismic load for shear wall line B is divided by 2 for archetype design since two shear walls are assumed along the wall line. This effective doubling of shear wall length along the wall line, which is impractical in some cases, was used in lieu of redesigning index buildings that provided inadequate wall length for resisting the full tributary seismic shear forces.

Table B150—Archetype 30 (9_B_6 _1_HR_HG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.24	2.27	483.3	10.42	4.83	2.16
5	4	M	2.5	4.125	3	A3	3	S	1563.0	4	3/4	0.45	2.94	1202.8	15.63	12.03	1.30
4	4	M	2.5	4.125	3	A3	2	D	2084.0	4	1	0.58	3.40	1771.8	20.84	17.72	1.18
3	4	M	2.5	4.125	3	A3	3	D	3126.0	4	1 3/8	0.60	4.04	2192.2	31.26	21.92	1.43
2	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 3/4	0.52	5.22	2466.7	31.26	24.67	1.27
1	4	M	2.5	6.875	5	A3	3	D	3126.0	4	2	0.55	5.24	2599.0	31.26	25.99	1.20

*Values shown are for reinforced option and before applying C_d.

Table B151—Archetype 42 (9_B_6 _1_ HR_LG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.23	2.27	472.5	10.42	4.73	2.21
5	4	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.54	2.11	1024.7	10.42	10.25	1.02
4	4	M	2.5	4.125	3	A3	3	S	1563.0	4	5/8	0.63	2.59	1461.3	15.63	14.61	1.07
3	4	M	2.5	6.875	5	A3	2	D	2084.0	4	3/4	0.57	3.47	1783.9	20.84	17.84	1.17
2	4	M	2.5	6.875	5	A3	2	D	2084.0	4	7/8	0.63	3.54	1994.5	20.84	19.95	1.04
1	4	M	2.5	6.875	5	A3	3	D	3126.0	4	1 1/8	0.53	4.39	2096.1	31.26	20.96	1.49

*Values shown are for reinforced option and before applying C_d.

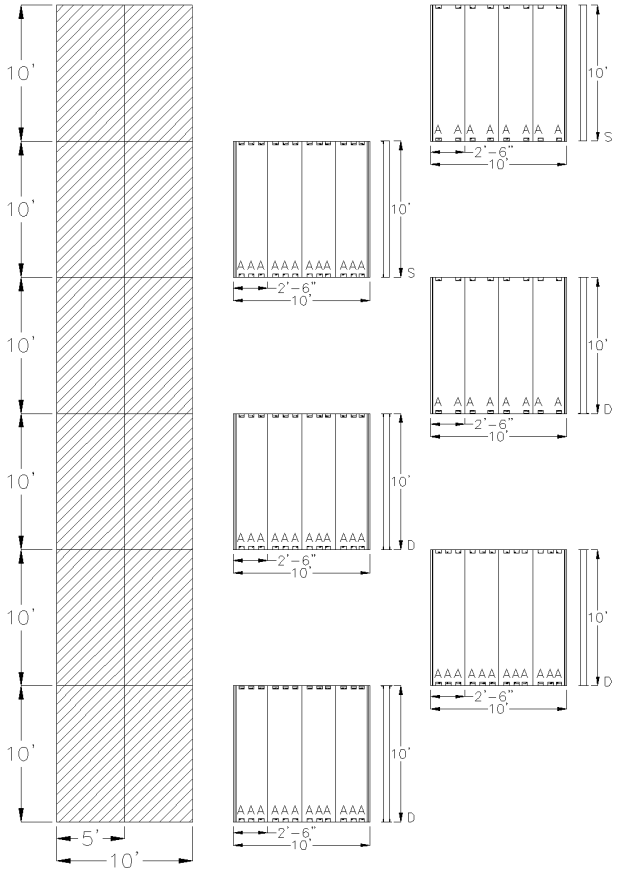


Figure B158. Archetype 30.

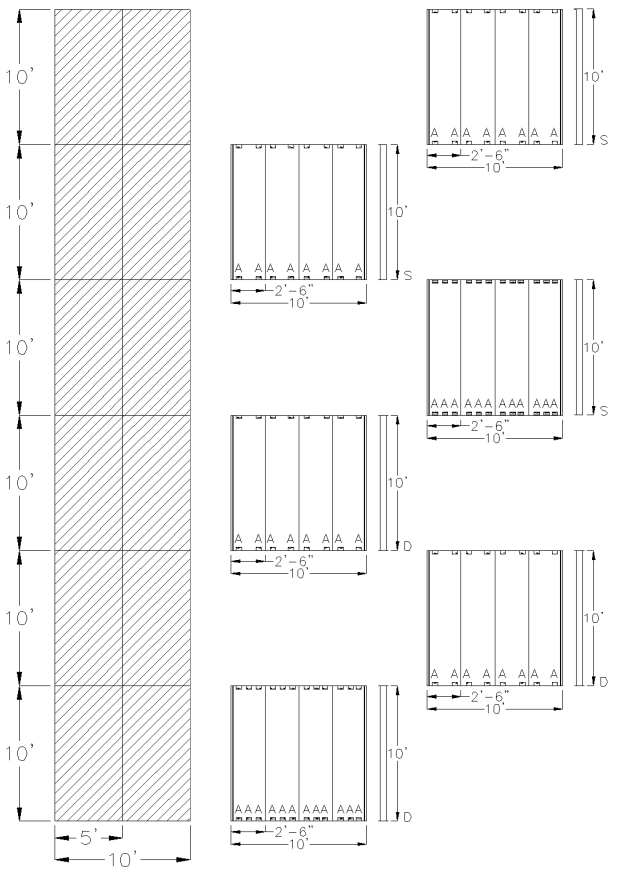


Figure B159. Archetype 42.

Table B152—Archetype 102 (9_3_6 _2_ HR_HG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.13	11.56	305.6	46.89	13.75	3.41
5	18	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.33	11.58	760.5	46.89	34.22	1.37
4	18	M	2.5	4.125	3	A3	3	S	1563.0	4	7/8	0.38	14.57	1120.2	70.34	50.41	1.40
3	18	M	2.5	6.875	5	A3	3	S	1563.0	4	1	0.40	17.34	1386.0	70.34	62.37	1.13
2	18	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/8	0.45	17.33	1559.5	70.34	70.18	1.00
1	18	M	2.5	6.875	5	A3	2	D	2084.0	4	1 3/8	0.40	20.46	1643.2	93.78	73.94	1.27

*Values shown are for reinforced option and before applying C_d.

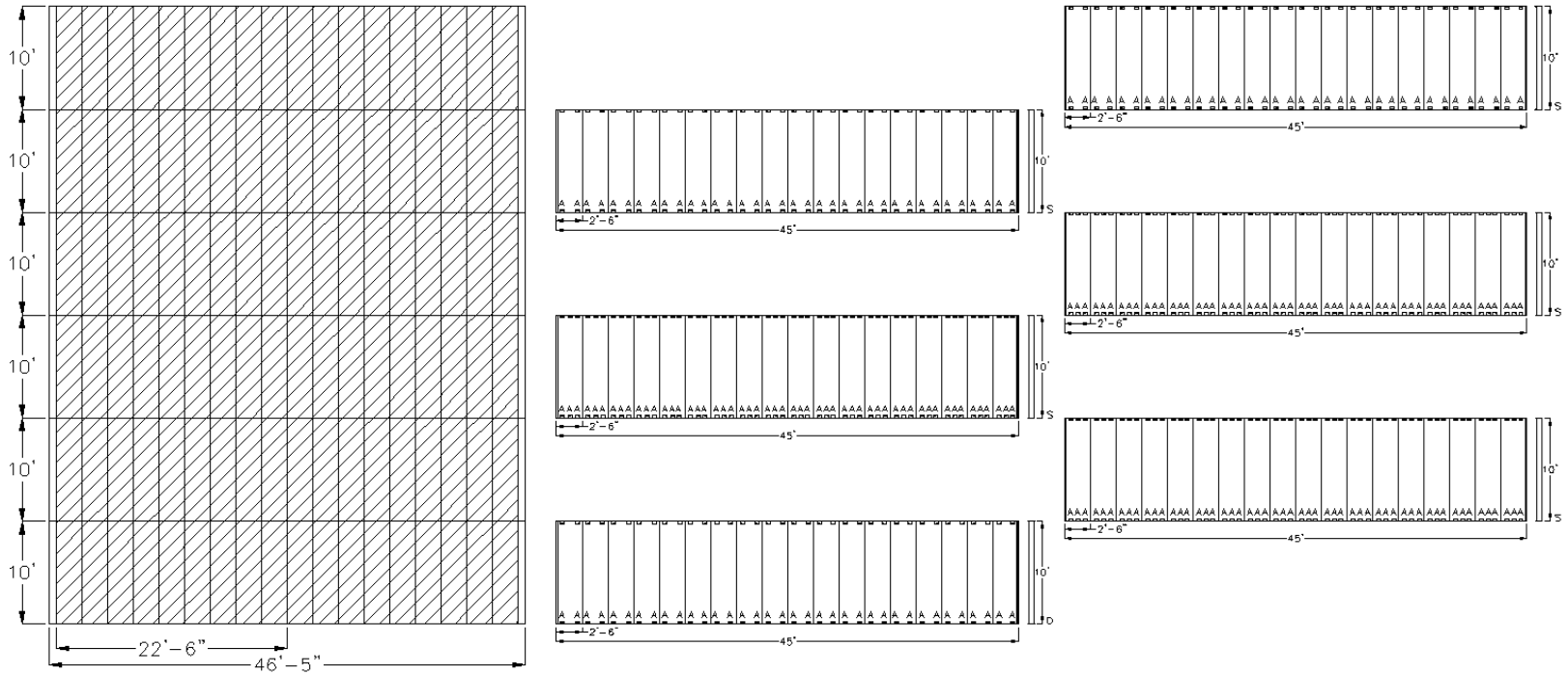


Figure B160. Archetype 102.

Table B153—Archetype 114 (9_3_6 _2_ HR_LG_ DX_LP) design, R = 4

Story	No. of Panels	Configuration	Panel Length (ft)	Thickness (in.)	No. of Layers	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (lb/ft)	No. of rods	Dia. (in.)	Deflection (in.)*	Stiffness (kip/in.)	Applied Load (lb/ft)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	4.125	3	A3	2	S	1042.0	4	1/2	0.13	11.56	298.8	46.89	13.44	3.49
5	18	M	2.5	4.125	3	A3	2	S	1042.0	4	5/8	0.28	11.56	647.8	46.89	29.15	1.61
4	18	M	2.5	4.125	3	A3	2	S	1042.0	4	3/4	0.40	11.56	923.9	46.89	41.57	1.13
3	18	M	2.5	4.125	3	A3	3	S	1563.0	4	1	0.39	14.54	1127.9	70.34	50.75	1.39
2	18	M	2.5	4.125	3	A3	3	S	1563.0	4	1 1/8	0.43	14.53	1261.0	70.34	56.75	1.24
1	18	M	2.5	6.875	5	A3	3	S	1563.0	4	1 1/4	0.38	17.31	1325.2	70.34	59.64	1.18

*Values shown are for reinforced option and before applying C_d.

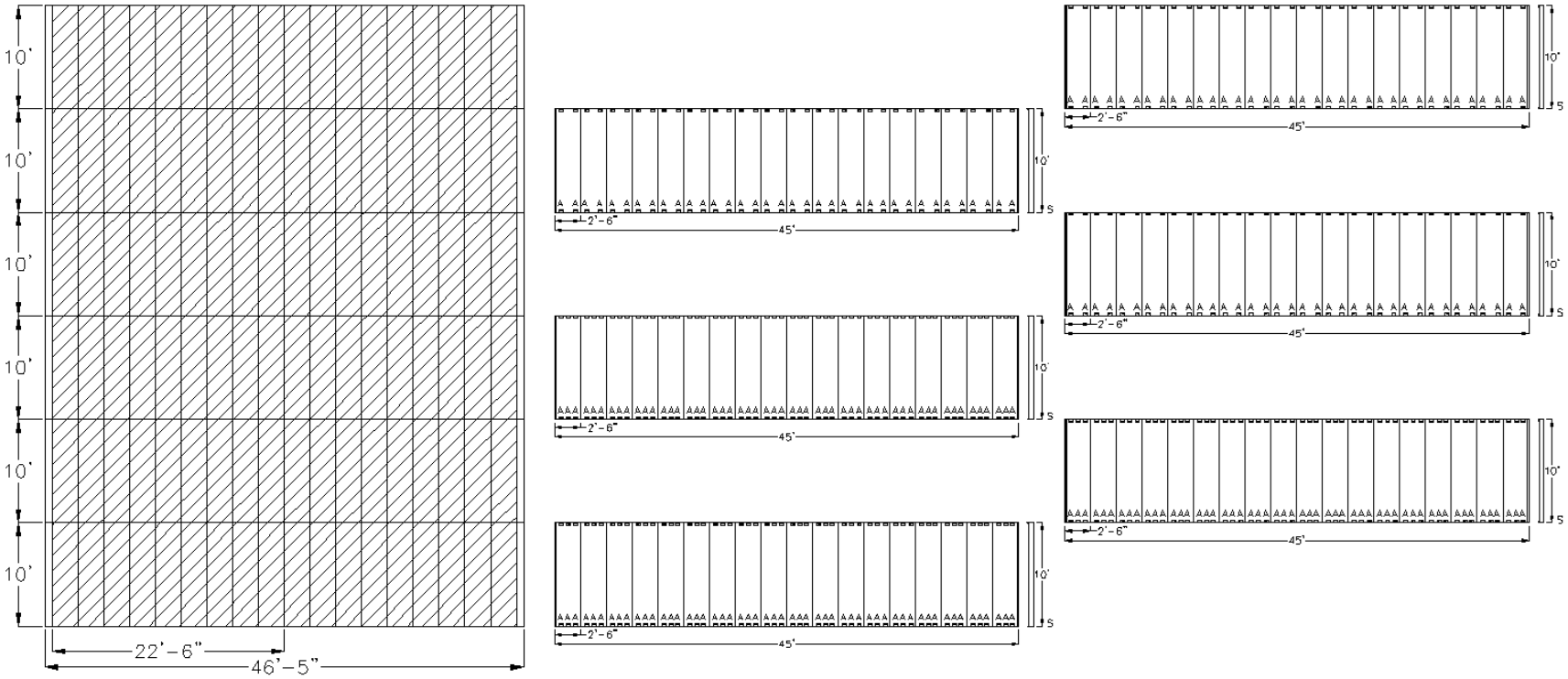
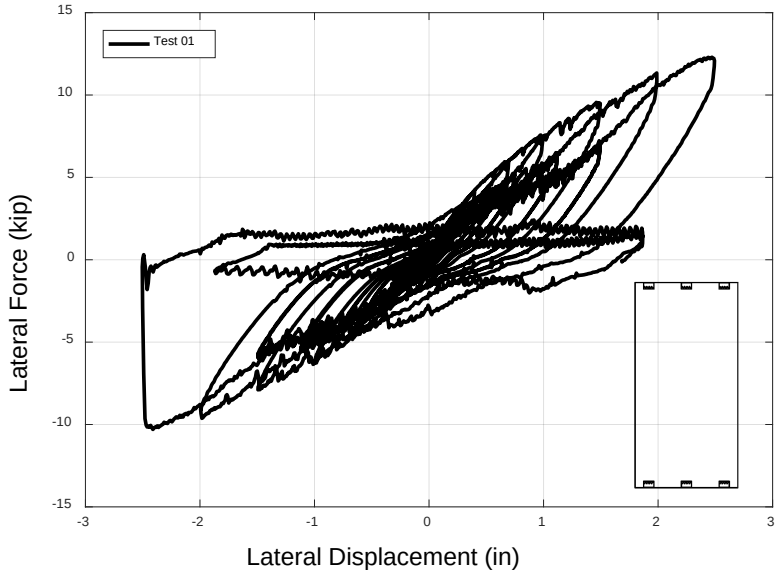


Figure B161. Archetype 114.

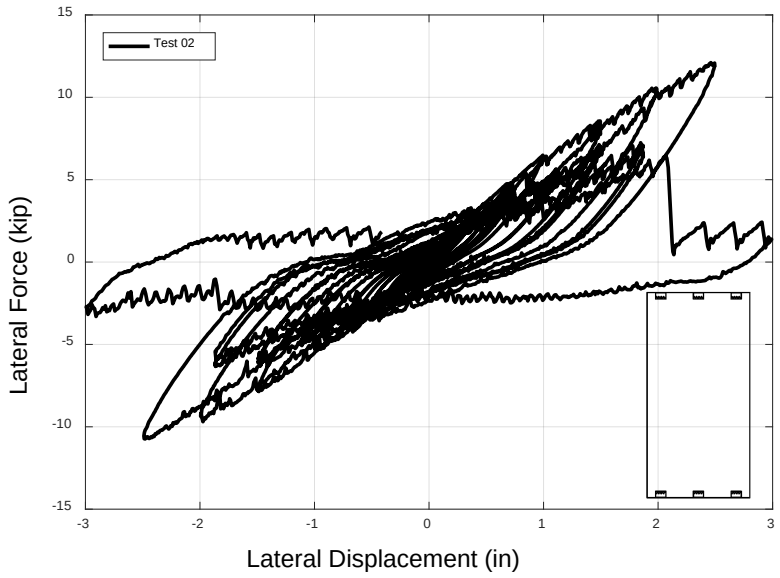
Appendix C—CLT Shear Wall Test Results

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
01	V2	8	4	5	3.9	3	A1	0.68
02	V2	8	4	5	3.9	3	A1	0.68



(b)

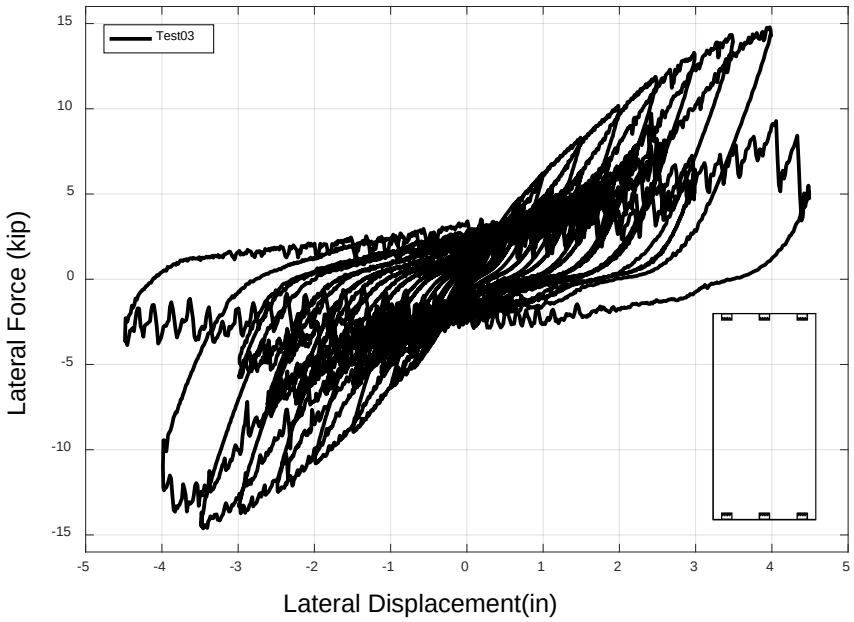


(c)

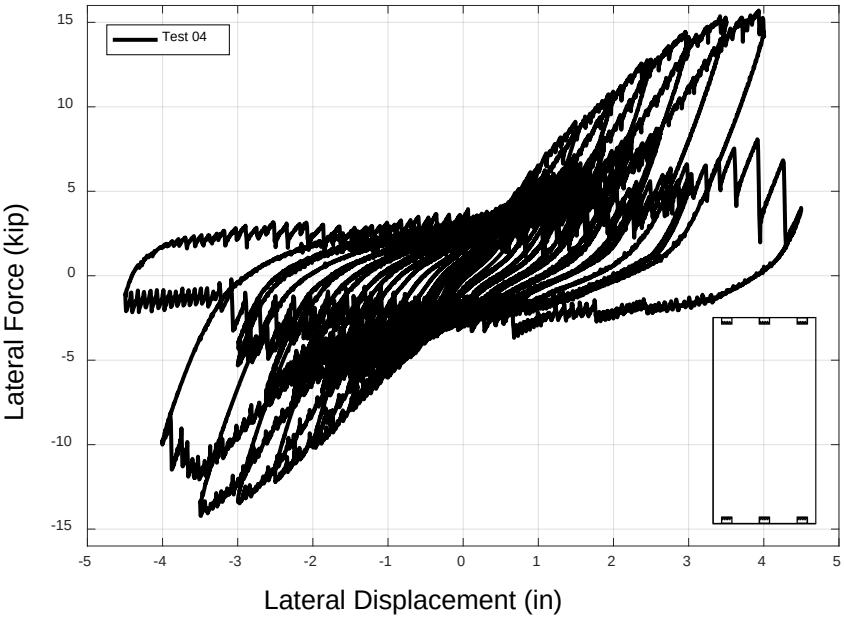
Figure C1. Tests 1 and 2: (a) test information; (b) Test 1 hysteresis; (c) Test 2 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
03	V2	8	4	5	6.65	3	A3	0.68
04	V2	8	4	5	6.65	3	A3	1.28



(b)

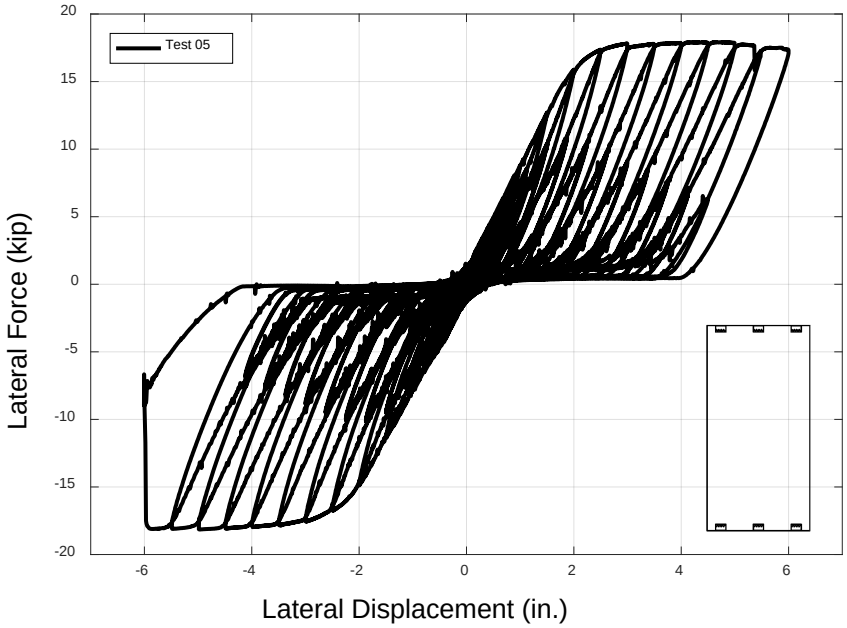


(c)

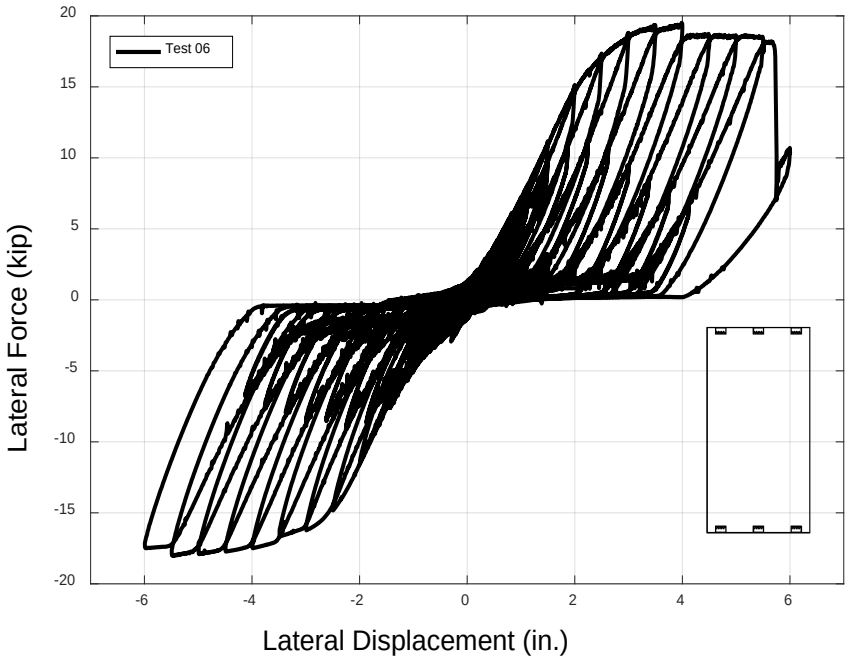
Figure C2. Tests 3 and 4: (a) test information; (b) Test 3 hysteresis; (c) Test 4 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
05	E1	8	4	5	6.89	3	A3	0.68
06	E1	8	4	5	6.89	3	A3	0.68



(b)

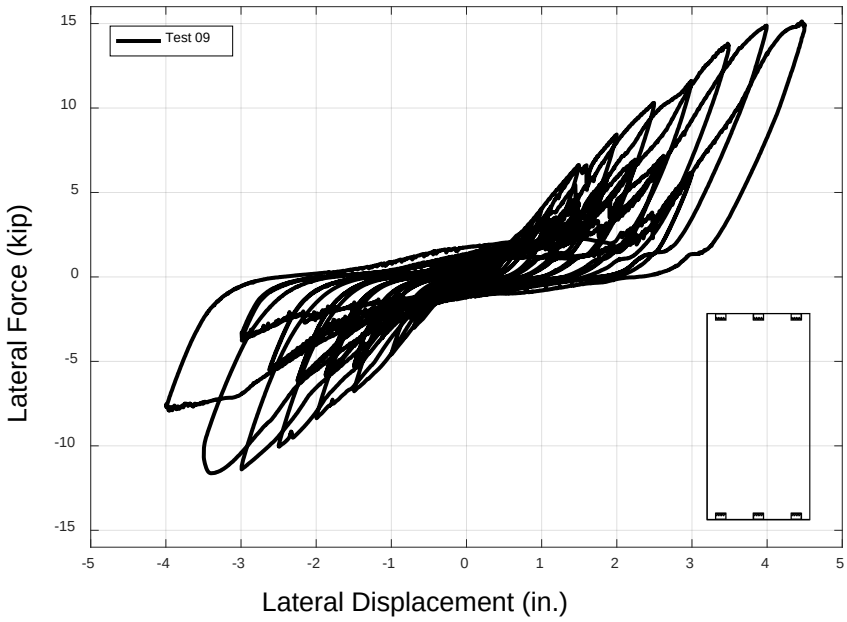


(c)

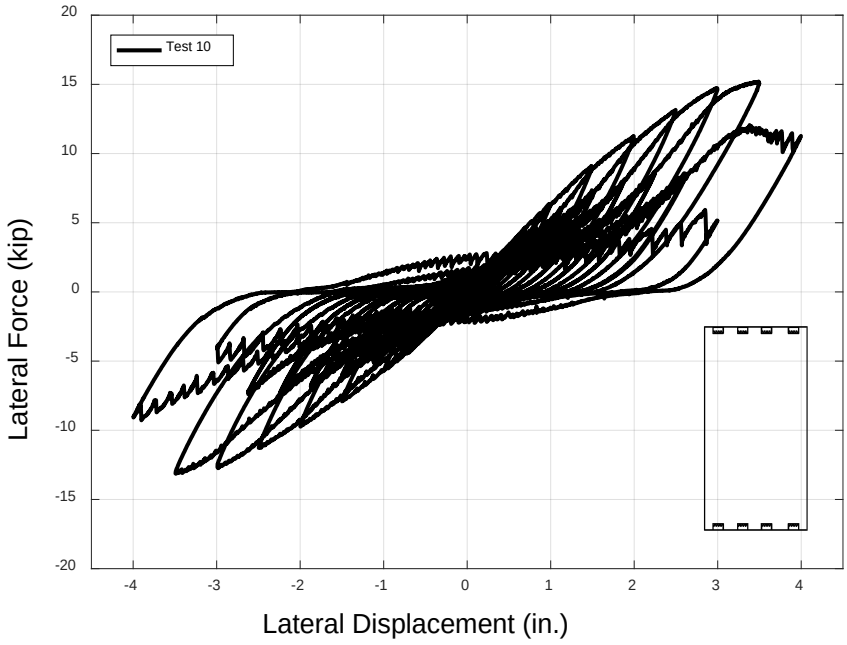
Figure C3. Tests 5 and 6: (a) test information; (b) Test 5 hysteresis; (c) Test 6 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
09	V2	8	4	5	6.65	3	A3	-
10	V2	8	4	3	3.9	4	A3	-



(b)

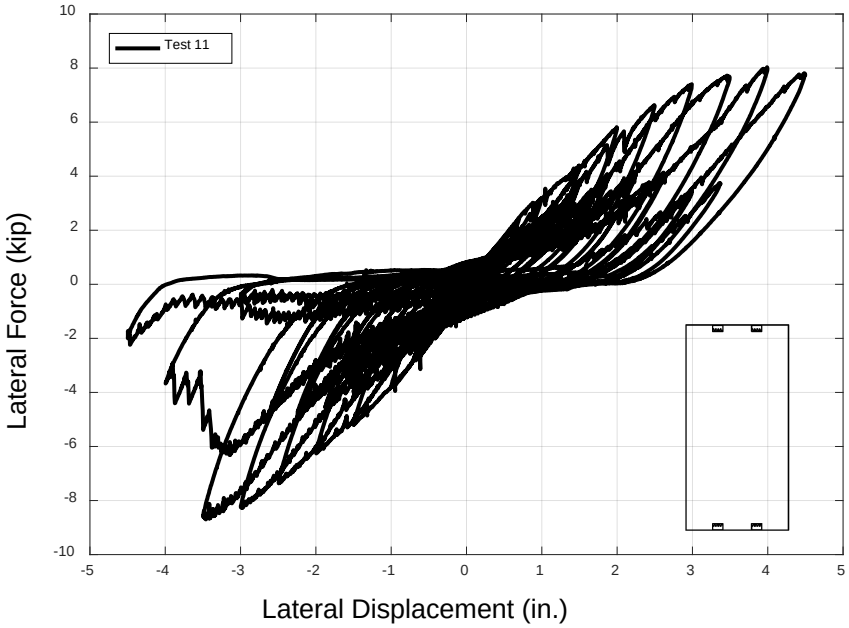


(c)

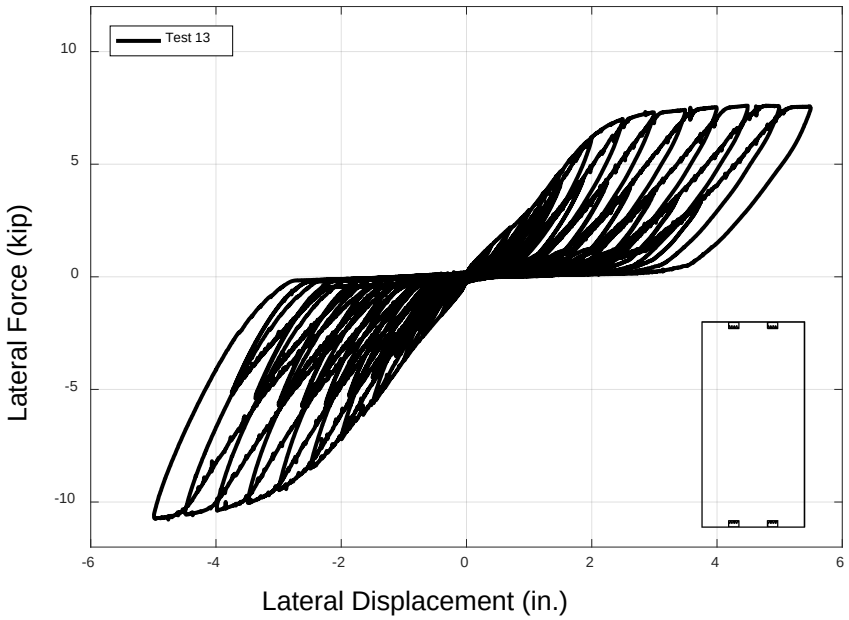
Figure C4. Tests 9 and 10: (a) test information; (b) Test 9 hysteresis; (c) Test 10 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
11	V2	8	4	5	6.65	2	A3	-
13	E1	8	4	5	6.89	2	A3	-



(b)

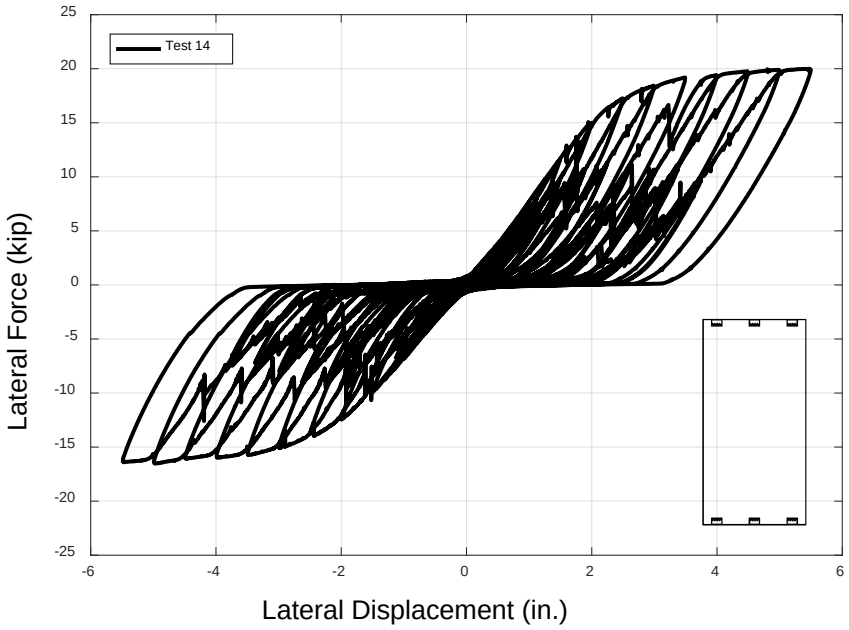


(c)

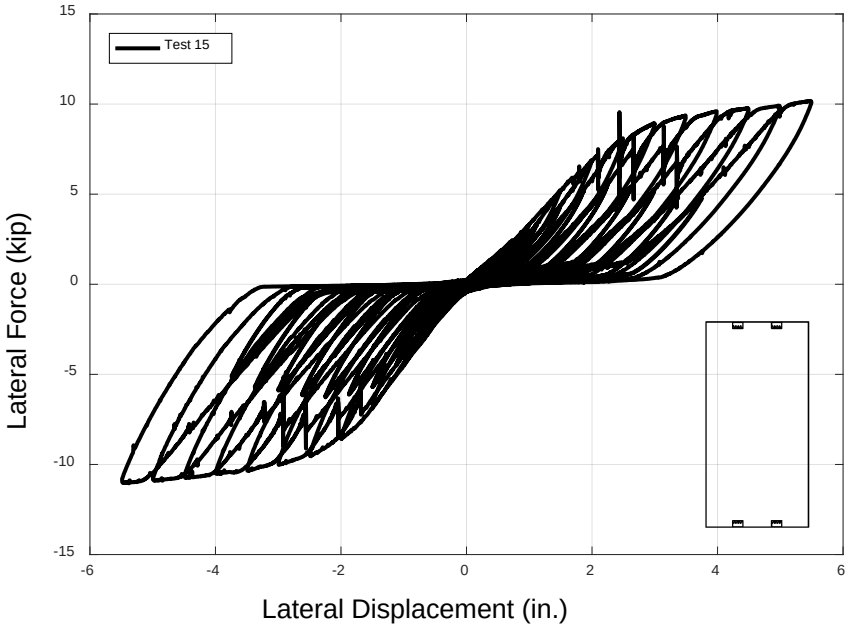
Figure C5. Tests 11 and 13: (a) test information; (b) Test 11 hysteresis; (c) Test 13 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
14	E1	8	4	5	6.89	3	A3	-
15	E1	8	4	5	6.89	2	A3	-



(b)



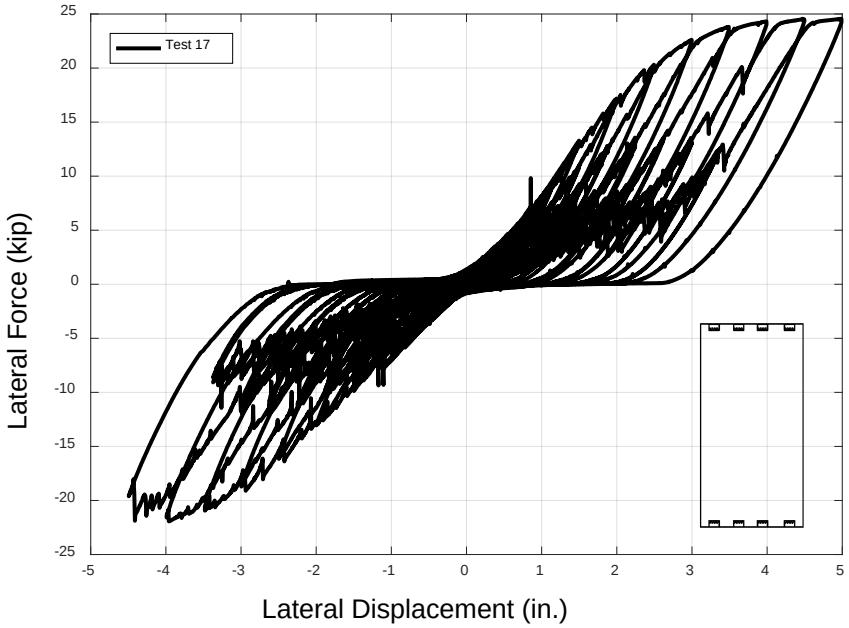
(c)

Figure C6. Tests 14 and 15: (a) test information; (b) Test 14 hysteresis; (c) Test 15 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
17	E1	8	4	5	6.89	4	A3	-
18	V2	8	4	3	3.9	2	A3	-

(b)



(c)

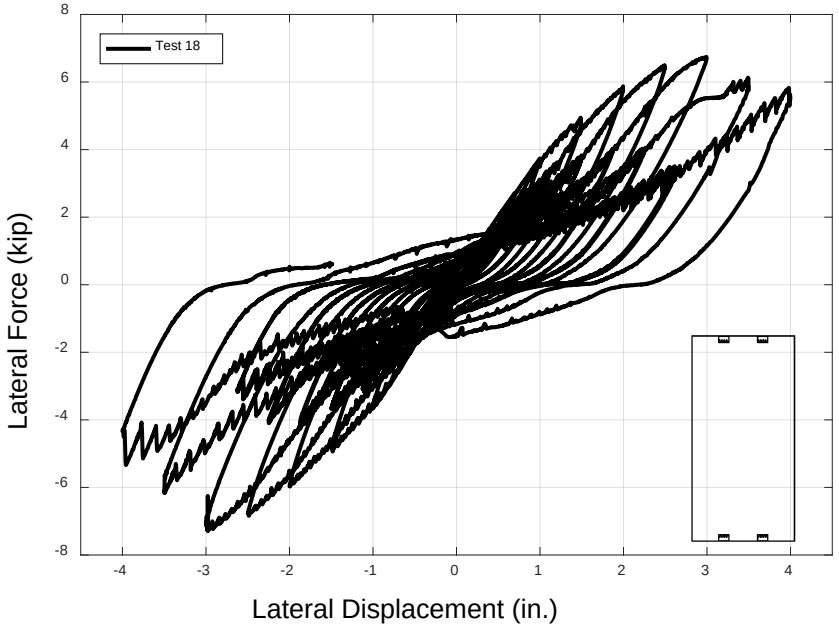
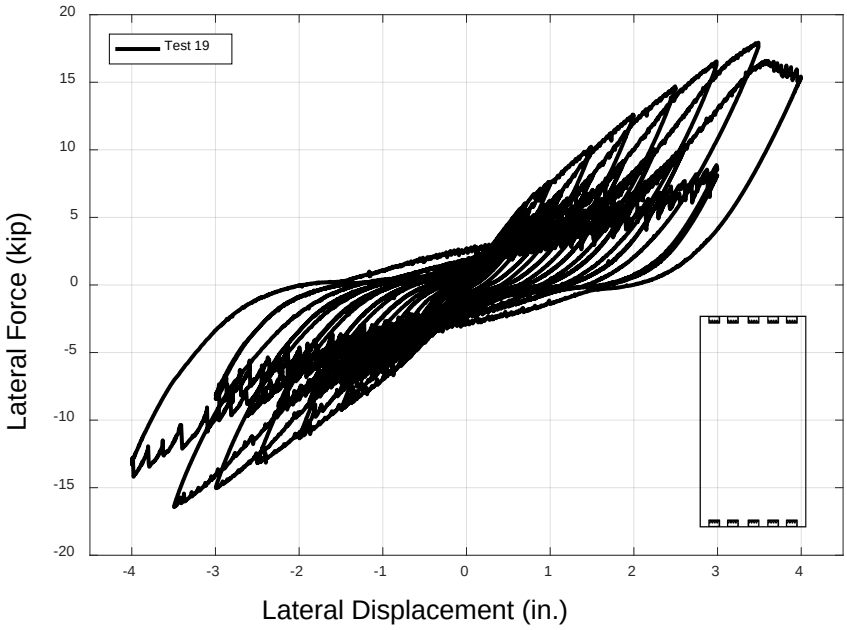


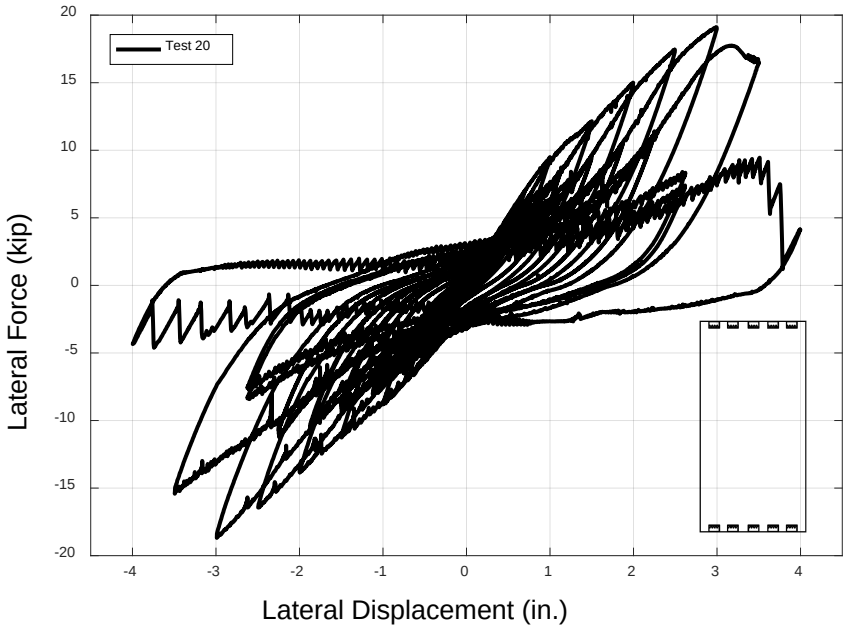
Figure C7. Tests 17 and 18: (a) test information; (b) Test 17 hysteresis; (c) Test 18 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
19	V2	8	4	3	3.9	5	A3	-
20	V2	8	4	7	9.41	5	A3	-



(b)

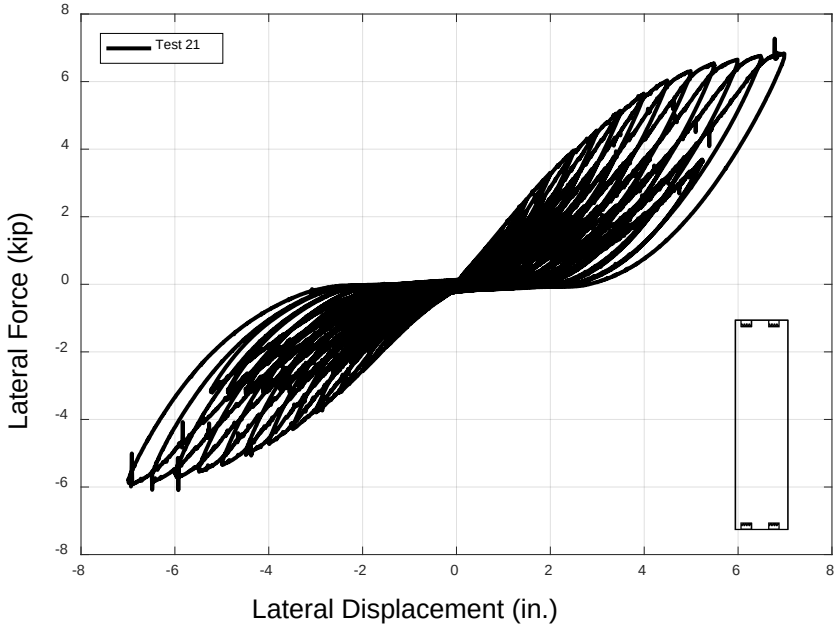


(c)

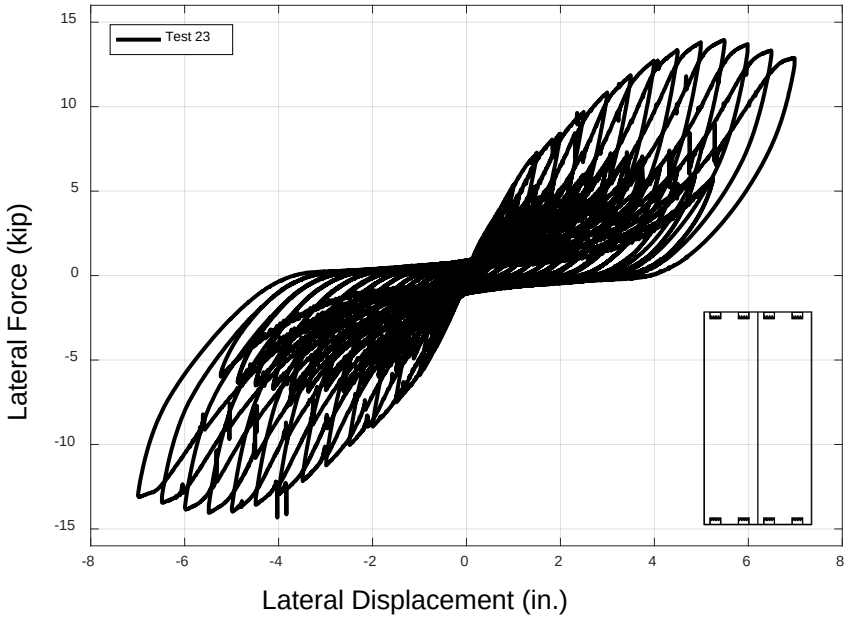
Figure C8. Tests 19 and 20: (a) test information; (b) Test 19 hysteresis; (c) Test 20 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
21	V2	8	2	3	3.9	2	A3	-
23	V2	8	2 (2)	5	6.65	4	A3	-



(b)

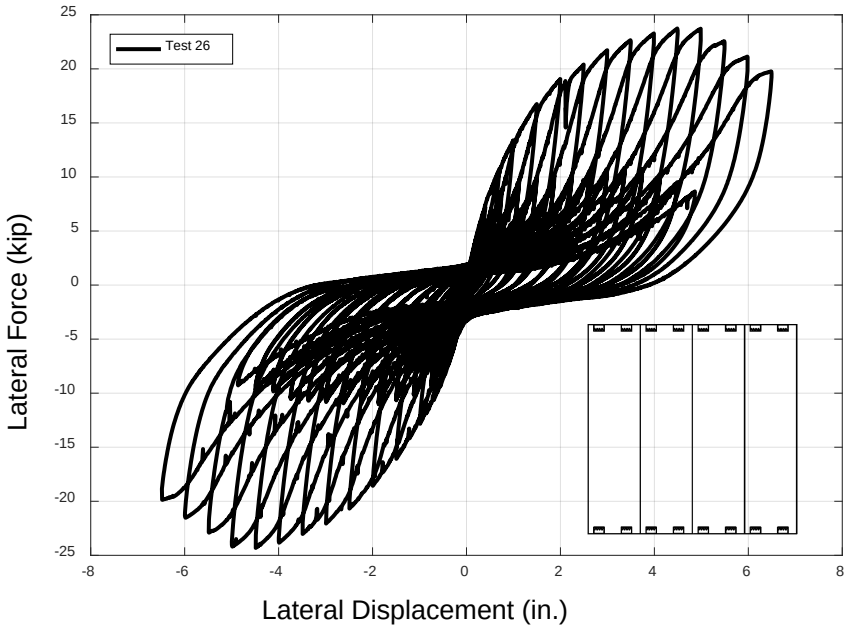


(c)

Figure C9. Tests 21 and 23: (a) test information; (b) Test 21 hysteresis; (c) Test 23 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

(a)

Test number	Grade	Height (ft)	Length (ft)	Number of plys	Thickness (in.)	No. connectors	Connector type	Gravity load (kip/ft)
26	V2	8	4 (2)	5	6.65	8	A3	-



(b)

Figure C10. Test 26: (a) test information; (b) Test 26 hysteresis (1 ft = 0.3 m; 1 in. = 25.4 mm; 1 kip/ft = 14.6 kN/m; 1 kip = 1,000 lb = 0.45 metric ton).

Appendix D—Design Methodology Commentary

C-5.1 Scope

Cross-laminated timber (CLT) shear wall design requirements are intended to produce yielding of nails and metal connectors at CLT panel edges and combined rocking and sliding behavior of individual wall panels prior to occurrence of the ultimate shear wall strength limit state associated with nailed connection failure. CLT shear walls can be in single-panel or multipanel configurations. Design unit shears are associated with uniform spacing of prescribed connectors at the bottom of the panel, at the top of the panel, and at vertical edges of multipanel shear walls. Typical single-panel and multipanel wall configurations are shown in Figure D1, and examples of typical connection details are depicted in Appendix E. Although angle connectors are shown at the top and bottom on only one face of the CLT wall panel, it is permissible to place connectors on both faces. The minimum requirement of two connectors per panel is still satisfied with connectors on opposite faces of the CLT wall panel.

Multipanel shear walls are formed with individual panels that have the same aspect ratio, which promotes deformation compatibility within the shear wall. The design requirements produce yielding of the prescribed nailed connections and rocking behavior in the shear wall as depicted in Figure D2 when subjected to in-plane shear forces. Details of Appendix E do not incorporate concrete floor toppings for clarity of illustrating connector requirements for in-plane shear loading and added fastening for out-of-plane loads. A clear space should be provided between such toppings and the vertical and horizontal legs of the connector to avoid inhibiting connector deformation (e.g., bending, tension, and rotation) under in-plane shear loading of the shear wall.

C-5.2 Application Requirements

The CLT shear wall lateral force resisting system (LFRS) is intended for use in platform construction in which all individual wall panels are single-story clear-height panels and the CLT floor panels are designated as the floor diaphragm. Elements of the gravity framing system can include but need not be limited to CLT walls, beams and columns, and light-frame walls. The required use of the CLT shear wall LFRS in platform construction precludes application for balloon frame construction associated with multistory clear-height wall panels. For gable end wall conditions, the requirement for wall panels of the same height necessitates a configuration of wall panels of the same height in the story below the gable end while the “triangular” gable end wall portion can be composed of CLT wall panels or other elements designed as the collector.

The design method requires similar detailing (i.e., minimum panel aspect ratio and shear connections) for all CLT wall panels whether part of the designated LFRS or not to promote deformation compatibility with the CLT shear wall system up to the point of failure. CLT wall panels that are not part of the designed LFRS are expected to be present in addition to the CLT shear walls designed as the vertical elements of the LFRS. In general, such added wall elements are considered to decrease in-plane shear demands on the LFRS and improve the strength and stiffness of the building as a whole, much like the presence of sheathed walls in excess of the designed shear walls in a wood-frame shear wall structure. However, such added wall elements may produce adverse effects on the structural system response

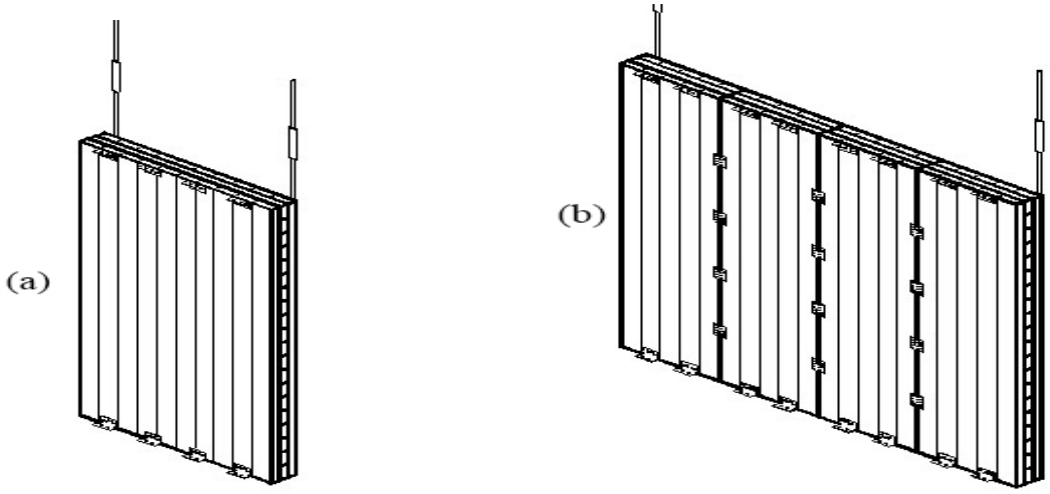


Figure D1. Typical shear wall for (a) single-panel configuration and (b) multipanel configuration.

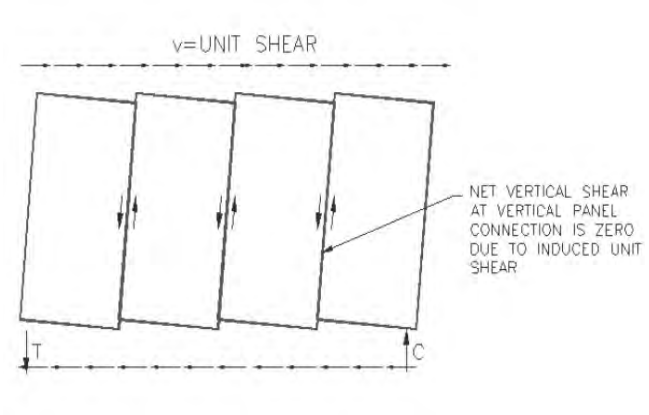


Figure D2. Rotation of individual panels in CLT shear wall.

that must also be considered in design of the structural system, including but not limited to the distribution of forces and load path to elements of the structural system, which may require strengthening relative to a design ignoring the interaction with CLT wall panels that are not designed as shear walls. In addition, consideration must be given to the potential for CLT wall panels not designed as shear walls to create structural irregularities such as a weak story irregularity, torsional irregularity, in-plane discontinuity in vertical lateral force-resisting element irregularity, and out-of-plane offset irregularity.

A suggested method for evaluating the structure for the presence of ASCE 7 (ASCE 2016) structural irregularities involves consideration of two separate cases representing bounding values of strength and stiffness contributed by the CLT panels that are not part of the designated seismic force resisting system (SFRS): (1) considering elements that are part of the designated SFRS alone, and (2) considering elements not part of the designated SFRS in combination with the elements that are part of the designated SFRS. Per this method, the bounding values of strength for CLT panels that are designated as not part of the SFRS range from a minimum of zero in Case 1 to a maximum equal to shear wall strength associated with full overturning restraint in Case 2. For some structural irregularities, placement criteria rather than structural distribution of strength and stiffness can trigger an irregularity (e.g., out-of-plane offset, in-plane discontinuity, and nonparallel system irregularity). Such irregularities can be triggered for CLT wall panels whether part of the designated SFRS or not part of the designated SFRS.

CLT wall panels with the prescribed nailed connectors are expected to contribute strength and stiffness across the full range of displacement expected of the CLT shear walls as seen in testing results of CLT shear walls with similar nailed connectors with and without hold-downs (Karacabeyli and Douglas 2013). The extent to which CLT wall panels that are not part of the designated LFRS add strength and stiffness depends on the level of overturning restraint provided to the individual wall panels through dead load and overturning restraint by surrounding elements. The design requirements conservatively prescribe that the strength and stiffness contribution of such walls, for purposes of determining their adverse effect on the structural system, be taken as equal to a shear wall with full overturning restraint provided at wall ends. It is recognized that alternative design approaches may entail detailing of CLT wall panels to either isolate them from resisting in-plane shear forces or to minimize their resistance to in-plane shear forces (such as through use of slotted holes to promote sliding and/or rocking) while also providing equivalent deformation capacity to the prescribed in-plane shear connections. Where such alternative approaches are used, effects of the alternative design for CLT wall panels that are not part of the LFRS (including those associated with strength and stiffness of the alternative design elements) must be considered in the design of the structural system.

Loads are distributed to shear walls within the wall line based on the stiffness determined using Equation (5.1) for each shear wall within the wall line. For distribution of shear to vertical elements of the LFRS, a diaphragm can be idealized as flexible, idealized as rigid, or modeled as semirigid in accordance with the requirements of ASCE 7 Section 12.3 and the Special Design Provisions for Wind and Seismic (SDPWS).

C-5.3 CLT Shear Wall Requirements

CLT shear wall requirements include use of CLT panels of prescribed aspect ratios; use of prescribed nailed connectors at bottoms of panels, tops of panels, and adjoining vertical edge(s) of multipanel shear walls; a minimum required capacity for overturning tension devices; and compression zone length requirements.

The prescribed angle connectors and connectors at the adjoining vertical edge of multipanel shear walls have been evaluated under fully reversed cyclic testing of shear walls and should not be modified or substituted without verification of equivalent shear wall performance by cyclic testing of shear walls that evaluates simultaneous uplift and shear loading of the connectors. For the prescribed angle connector, observed failure from shear wall testing was caused by combined nail bending, nail withdrawal from the wood, and limited occurrence of combined bending–tension failure of the nail without metal connector failure. The prescribed connectors for the vertical edge provide equivalent shear capacity to that of the angle connectors at the top and bottom of the CLT shear wall. The prescribed connectors have been tested as part of a shear wall and as individual components under uplift loading and shear loading. For the testing, bolts were used in the horizontal leg of the connectors. Lag screws are prescribed as an

alternative with strength in shear and uplift capable of developing the tested strength of the nailed connector.

Design of CLT shear walls and associated load paths is in accordance with the basic load combinations of ASCE 7 (load combinations without overstrength) except where otherwise required by the National Design Specification (NDS). Hold-down requirements are intended for two common hold-down systems – continuous tie-rod systems and conventional hold-down devices. For both, the required design for two times the forces associated with the design unit shear capacity of the CLT shear wall is intended to ensure shear strength in excess of the specified nominal strength of the shear connections and is consistent with level of overstrength in the hold-down system in CLT shear wall testing. A device elongation limit of 0.185 in. for strength design is required to be met at each level to avoid concentration of device elongation in one level and is based on ICC-ES (ICC-ES 2010) evaluation criteria limit for continuous tie-rod systems used with wood-frame shear walls and for conventional hold-down devices attached to wood members.

Under in-plane unit shear loading, individual CLT panels within a CLT shear wall designed and detailed in accordance with Chapter 5 will rotate as shown in Figure D2. For purposes of determining tension force, *T*, and compression force, *C*, static equilibrium is based on consideration of the tension end panel and compression end panel depicted in Figure D3. Consistent with individual panel rotation behavior as opposed to overturning as a rigid body whole, the static analysis of individual panels is employed in determination of *T* and *C* forces. The contribution of dead load in the overturning design is specifically limited to only that dead load tributary to the individual panel and to elements aligned directly above the panel of interest per Figure D3. The dead load includes reactions from headers, beams, and similar elements when they are supported by the panel of interest. Vertical load reactions from floors above are to be applied to each panel of interest as *C* and *T* reactions for end panels and *C* reactions for interior panels, as applicable. As a result of this assumption of individual panel overturning, the overturning induced tension force is larger and overturning induced compression force is smaller than *T* and *C* forces associated with overturning the wall as a rigid monolith, primarily because the static analysis does not assume that distributed gravity loading over the length of the wall can be mobilized via a whole-wall rigid body assumption to decrease the *T* force or increase the *C* force.

For the tension end panel depicted in Figure D3, the tension force from summation of moment about point O is

$$\sum M_{\mathrm{O}} = 0$$

$$T \times b_{\mathrm{eff}} - \left(v \times b_{\mathrm{s}} \times h\right) + w \times b_{\mathrm{s}} \times \left(\frac{b_{\mathrm{s}}}{2}\right) - T_{\mathrm{T}} \times b_{\mathrm{eff}} = 0 \quad \text{(D1)}$$

$$T = \left[\left(v \times b_{\mathrm{s}} \times h\right) - w \times b_{\mathrm{s}} \times \left(\frac{b_{\mathrm{s}}}{2}\right)\right] / b_{\mathrm{eff}} + T_{\mathrm{T}} \quad \text{(D2)}$$

where

v is unit shear, lb/ft,

w is dead load including wall panel self-weight, lb/ft,

b_{s} is CLT panel length, ft,

b_{eff} is moment arm for calculation of *T* force, ft,

h is CLT panel height, ft,

T is tension force, lb, and

T_{T} is tension force at top of tension end panel from story above, lb.

For the compression end panel depicted in Figure D3, the compression force from summation of moment about O is

$$\sum M_{\mathrm{O}} = 0$$

$$C \times \left(b_{\mathrm{s}} - \frac{x}{2}\right) - \left(v \times b_{\mathrm{s}} \times h\right) - \left(w \times b_{\mathrm{s}} \times \frac{b_{\mathrm{s}}}{2}\right) - C_{\mathrm{T}}\left(b_{\mathrm{s}} - \frac{x_{\mathrm{T}}}{2}\right) = 0 \quad \text{(D3)}$$

where

v is unit shear, lb/ft,

w is dead load including wall panel self-weight, lb/ft,

b_{s} is CLT panel length, ft,

h is CLT panel height, ft,

C is compression force, lb,

b_{s} is CLT panel length, ft,

x is length of compression zone, ft,

C_{T} is compression force at top of compression end panel from story above, lb, and

x_{T} is length of compression zone at top of compression end panel from story above, ft.

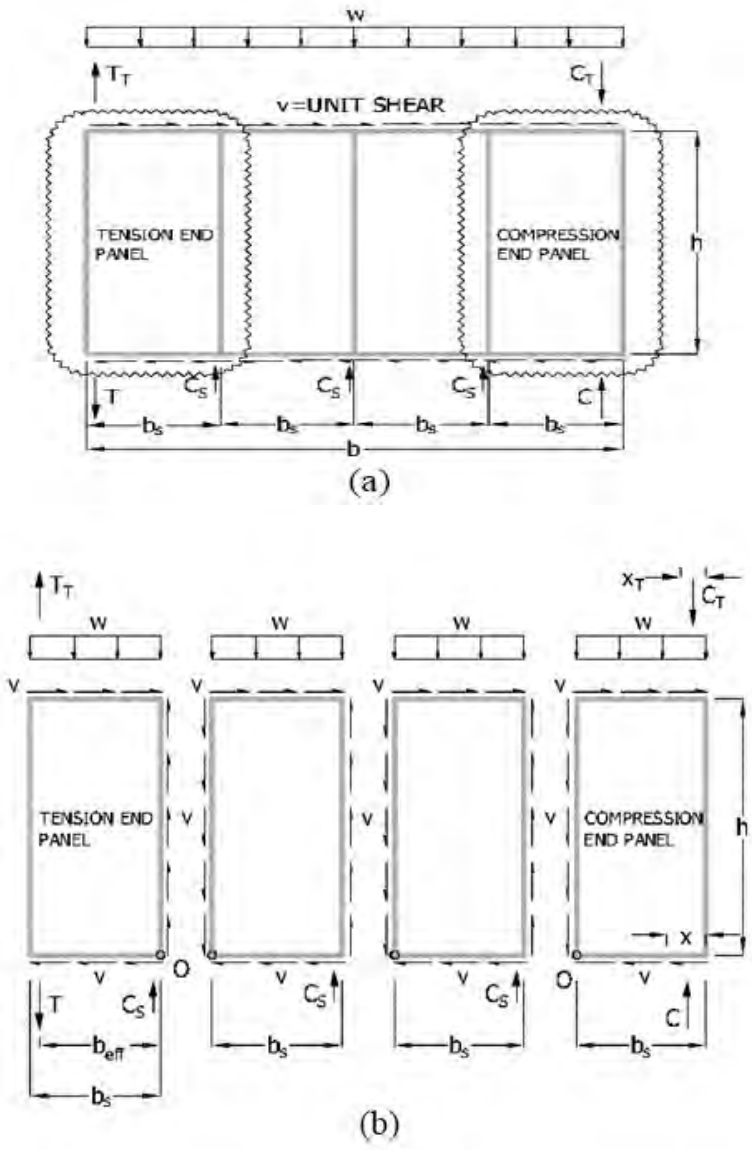


Figure D3. Combined shear and gravity loading and geometry for CLT shear wall composed of multiple CLT panels (multipanel shear wall): (a) compression end and tension end panels circled; (b) individual CLT panel overturning and opposing internal shears at adjacent vertical edges caused by in-plane unit shear loading.

The compression force associated with the length of the compression zone, assuming a uniform stress distribution (i.e., rectangular stress distribution) in the compression zone, is determined by the following equations:

$$C = F_{C\perp}'(t)(x)\left(\frac{12\text{ in.}}{\text{ft}}\right) \quad \text{(D4)}$$

$$C = F_C'\left(t_{\text{parallel}}\right)(x)\left(\frac{12\text{ in.}}{\text{ft}}\right) \quad \text{(D5)}$$

where

x is length of compression zone, ft,

C is compression force, lb,

$F_{C\perp}'$ is bearing stress perpendicular to grain, lb/in^2,

F_C' is design axial stress parallel to grain, lb/in^2,

t is CLT wall panel thickness, in.,

t_{parallel} is thickness of CLT wall panel layers oriented parallel to grain for determination of wall axial capacity, in., and

12 in./ft is conversion of compression zone length from feet to inches.

The bearing resistance in Equation (D4) is based on the compression perpendicular to grain stress associated with the CLT floor panel or wood bottom plate supporting the wall. The bearing resistance in Equation (D5) is based on the design axial stress associated with the CLT wall panel parallel to grain layers and is applicable where designed compression zone elements are used to transfer such forces as opposed to being limited by compression perpendicular to grain bearing stress in the floor panel. The designed compression zone force transfer detail through the floor panel is likely to be used in cases with a combination of high axial compression loads and high aspect ratio panels for the purpose of limiting wall thickness increases associated with meeting compression zone length requirements.

The length of the uniform stress compression zone, *x*, to satisfy static equilibrium is determined by substitution of Equation (D4) or (D5) into Equation (D3) and solving for *x*. The solution for *x* limited by bearing stress perpendicular to grain is provided in Equation (D6).

$$x = \frac{6F'_{C\perp}tb_s - \sqrt{36F_{C\perp}{}'^2tb_s^2 - 6F_{C\perp}{}'t\left[vb_sh + wb_s^2/2 + C_T\left(b_s - x_T/2\right)\right]}}{6F_{C\perp}{}'t} \quad \text{(D6)}$$

Where the length of the compression zone, *x*, is smaller than the length of the compression end panel, b_s, a positive value under the root in Equation (D6) is produced and the resulting value of *x* can be used to determine a precise value of compression force, *C*, in accordance with Equation (D3). A negative value under the root of Equation (D6) signifies the compression zone is not contained within the compression end panel. A preliminary check for whether adequate compression panel length is provided under unit shear loading alone (e.g., $w = C_T = 0$) can be obtained from Equation (D7). When Equation (D7) is not satisfied, a negative root will occur in Equation (D6), indicating inadequate compression panel length.

$$v \le \frac{6F_{C\perp}{}'t}{h/b_s} \quad \text{(D7)}$$

The loading and geometry depicted in Figure D3 for tension end and compression end panels are for purposes of illustrating a method to calculate appropriate *T* and *C* forces for the system. Testing showed that rotation of the compression end panel was primarily about the outermost edge of the compression end panel – not about the centroid of the calculated compression zone. As such, using the loading and geometry depicted in Figure D3 for the compression end panel will underestimate the moment arm and overestimate vertical edge forces when summing forces vertically at that location. Results of such analysis should not be used to modify required vertical connector spacing in accordance with Section 5.3.3 in Chapter 5, which requires the same average vertical connector spacing with rounding as used for the top and bottom edges of the CLT shear wall. The required number of connections at vertical and horizontal edges (i.e., the same average spacing) provides balanced vertical and horizontal shear and enables the intended rotation behavior of individual panels of a multipanel shear wall when subjected to in-plane unit shear loading.

C-5.3.6 Other Load Path Connections to CLT

Load path connections to CLT wall panels occur in addition to those of the designated LFRS for in-plane shear resistance and include connections for out-of-plane wind and seismic forces and general structural integrity. These additional load path connections include attachment of wall panels to elements above and below for out-of-plane forces, interconnection of walls at intersections, and attachment of conventional hold-down devices at wall ends.

The combined requirement for fastener yielding per Mode III_s or IV and compliance with NDS appendix E ensures that added fastening provides a predictable yielding mechanism with levels of overstrength similar to that provided by the prescribed connections for in-plane shear resistance. Top and bottom of wall connections for resistance to out-of-plane forces are in addition to prescribed angle connectors, which do not have an established design value for loads perpendicular to the plane of the wall. Top and bottom of wall connections meeting requirements for yielding in Mode III_s or IV are considered beneficial to in-plane shear wall strength and stiffness, which is already governed by nail yielding, without degrading peak load and post-peak response of the prescribed shear wall connectors.

To address the potential for excessive screw (e.g., wood and lag screws) attachment to inhibit the rocking mechanism of the CLT panel caused by high axial stiffness and strength of screws loaded in withdrawal, screw attachment of top and bottom of wall connections to supporting elements is not permitted. Screws used in these locations are considered an alternative to the prescribed smooth shank dowel fasteners (see Appendix E for typical connection details) at the top and bottom of wall locations and are subject to approval by the authority having jurisdiction. Typical fastening will employ smooth shank nails or pins to resist out-of-plane forces. Details for anchoring the top and bottom of walls for out-of-plane forces are not specifically prescribed to enable varying design options for meeting out-of-plane anchoring requirements.

C-5.3.7 CLT Shear Wall with Shear Resistance Provided by High Aspect Ratio Panels Only

CLT shear walls with shear resistance provided by high aspect ratio panels only is a specific configuration of the CLT shear wall system in which high aspect ratio is defined as wall panel height to wall panel length ratio of 4. Minor variations in actual panel aspect ratio of +/-2.5% percent are permissible such that the actual panel aspect ratio range is

3.9 to 4.1. All requirements applicable for CLT shear walls are also applicable and additionally only CLT panels with aspect ratio of 4 are permissible as part of the designated shear wall system. CLT wall panels of equal or greater aspect ratio are permissible when not used as part of the designated shear wall system to promote deformation compatibility of CLT wall panels that are not designed as shear walls. Where CLT shear walls with shear resistance provided by high aspect ratio panels only is used, it is required that the aspect ratio requirement be met in all shear walls. Although this limitation can be accommodated in single-story and multistory construction with equal story height, it may not be practical to implement where story height varies. In such cases, use of CLT shear walls with a permissible range in aspect ratio from 2 to 4 should be considered.

C-5.3 CLT Shear Wall Deflection

The CLT shear wall deflection equation incorporates four primary components: individual wall panel bending, individual wall panel shear, sliding, and rigid body overturning. Individual panel rotation is included for multipanel configurations. The deflection method accounts for the difference in observed stiffness of single- and multipanel CLT shear walls tested as well as the influence of individual panel aspect ratio on shear wall deflection. Components of shear wall deflection are depicted in Figure D4.

The bending term in the deflection equation is simplified from $vb_s h^3/(3 \times \mathrm{EI}_{\mathrm{eff}})$ for a cantilever with point load to $576vb_s h^3/(\mathrm{EI}_{\mathrm{eff}})$ to account for the unit conversion so that $\mathrm{EI}_{\mathrm{eff}}$ can be in lb-in^2 and other units can be in feet. $\mathrm{EI}_{\mathrm{eff}}$ is the effective in-plane panel stiffness for bending to account for composite behavior between adjacent parallel laminations where transverse E is approximated as longitudinal E/30. $\mathrm{EI}_{\mathrm{eff}}$ can be calculated directly using transformed section properties or with the following equation presented in Blass and Fellmoser (2004):

$$(\mathrm{EI})_{\mathrm{eff}} = \left[1-\left(1-\frac{E_{90,\mathrm{T}}}{E_{0,\mathrm{L}}}\right)\frac{a_{m-2}-a_{m-4}+\cdots\pm a_1}{a_m}\right]E_{0,\mathrm{L}} \times \frac{b_s^3 \times a_m}{12} \quad \text{(D8)}$$

where

$E_{0,\mathrm{L}}$ is modulus of elasticity parallel to the grain for longitudinal layers (i.e., longitudinal E),

$E_{90,\mathrm{T}}$ is modulus of elasticity perpendicular to the grain for transverse layers (i.e., transverse E taken as longitudinal E/30), and

a_1, a_3, a_5 are longitudinal layers of CLT as shown in the Figure D5 for a five-layer panel.

The panel shear deformation term uses in-plane shear stiffness, $\mathrm{GA}_{\mathrm{eff\ (in\text{-}plane)}}$, in units of pounds per inch (lb/in). Example values of $\mathrm{GA}_{\mathrm{eff\ (in\text{-}plane)}}$ provided in Table D1 are calculated in accordance with Flaig and Blass (2013) per Equations (D9) and (D10).

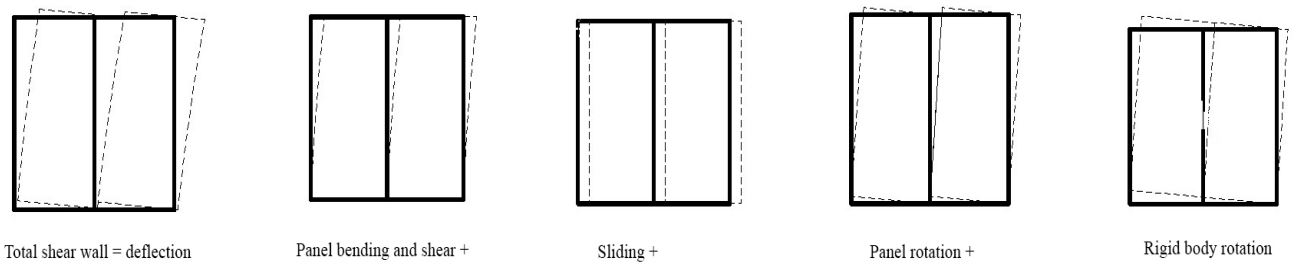


Figure D4. Shear wall deflection components: panel bending and shear, sliding caused by fastener slip, rotation caused by fastener slip at vertical edge connections, and rigid body rotation.

Table D1—Example $\mathrm{GA}_{\mathrm{eff}}$ (in-plane) for in-plane shear[a]

	Number of layers			
	3	5	7	9
G_{eff}, lb/in^2	28,700	32,300	33,700	34,500
$\mathrm{GA}_{\mathrm{eff\ (in\text{-}plane)}}$, lb/in^2	118,300	222,000	324,500	426,600

[a]Calculated values of G_{eff} and $\mathrm{GA}_{\mathrm{eff\ (in\text{-}plane)}}$ are based on use of Equations (D9) and (D10) with the following inputs: E = 1,400,000 lb/in^2, lamella width b = 5.5 in., number of longitudinal lamellae based on a 48-in. panel width, slip modulus of crossing areas K = 14,735 lb/in^3 (4.0 N/mm^3) and lamella thickness = 1.375 in.

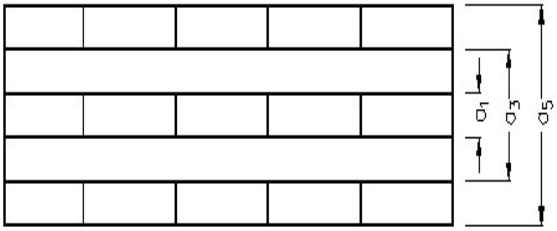


Figure D5. Illustration of a_1, a_3, and a_5.

$$G_{\text{eff, CA}} = \frac{Kb^2}{5} \cdot \frac{n_{\text{CA}}}{t_{\text{gross}}} \cdot \frac{m^2}{\left(m^2 + 1\right)} \quad \text{(D9)}$$

$$G_{\text{eff, CLT}} = \left(\frac{1}{G_{\text{lam}}} + \frac{1}{G_{\text{eff, CA}}} \right)^{-1} \quad \text{(D10)}$$

where

K is slip modulus of crossing areas (use K = 4.0 N/mm^3 = 14,735 lb/in^3),

b is width of lamella,

m is number of longitudinal lamellae,

n_{CA} is number of glue lines within CLT cross section thickness,

t_{gross} is CLT cross section thickness, and

G_{lam} is individual lamination shear modulus, lb/in^2 (use longitudinal E/16).

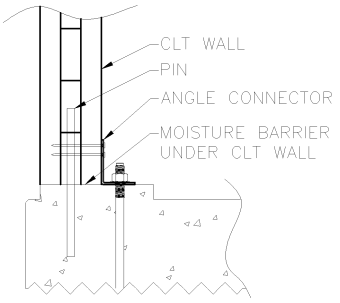
The sliding term, $V_{\text{nail load}}$/(135,000 D1.5), addresses sources of deformation in the connector including nails and bolts. The slip constant takes into account loading perpendicular to the grain in the nailed connection. The single nail diameter of 0.135 in. used for all the connectors in this study allows the use of a simplified nail slip term, $V_{\text{nail load}}$/(6,700). The deflection equation also explicitly breaks out sliding from multipanel rotation caused by vertical connection slip. If there is no vertical edge connection, then vertical connection slip equals 0 in. The final term in the deflection equation represents rigid body rotation about the compression toe of the shear wall and is the same as that used for sheathed wood-frame shear walls. Vertical deformation of the wall hold-down system, Δ_a, is based on the induced overturning forces and includes sources of deflection such as fastener slip, device elongation, rod elongation, uncompensated shrinkage, and vertical compression deformation.

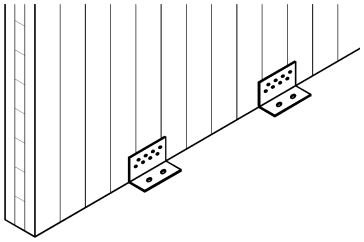
C-5.7 Diaphragm Requirements

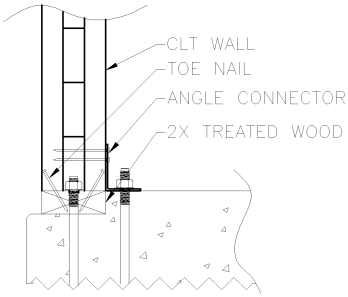
Fasteners used in diaphragm connections are required to be controlled by Mode III$_s$ or IV yielding. ASCE 7 Section 12.11 requirements are applicable to ensure wall-to-diaphragm integrity.

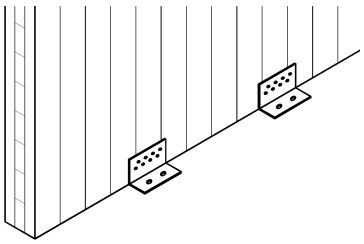
Appendix E—Detailing

Wall panel to foundation connection

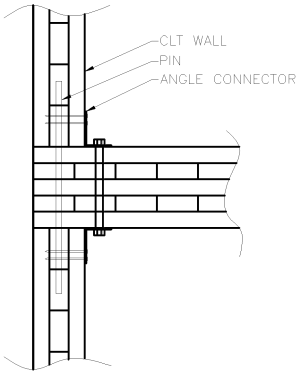


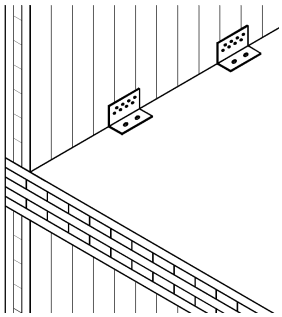


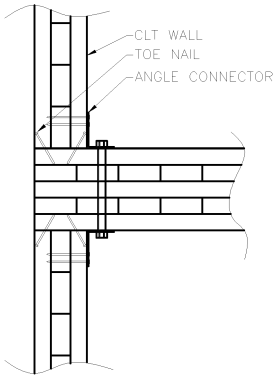


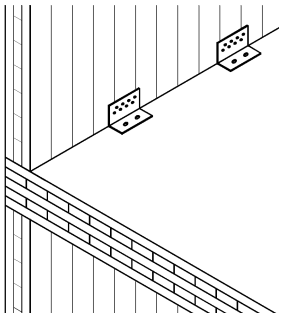


Exterior wall to floor connection

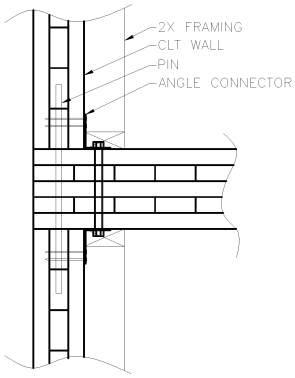


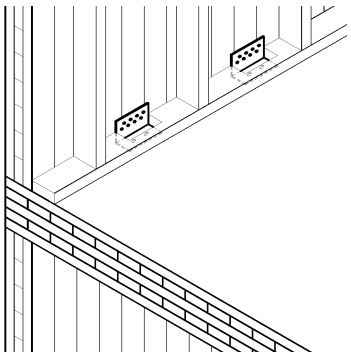




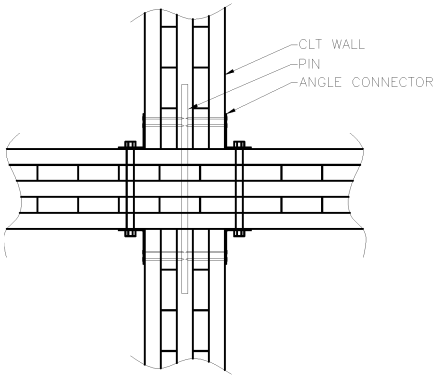


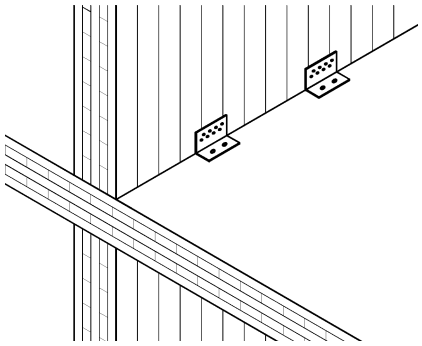
Exterior wall to floor connection (example with interior framed wall)

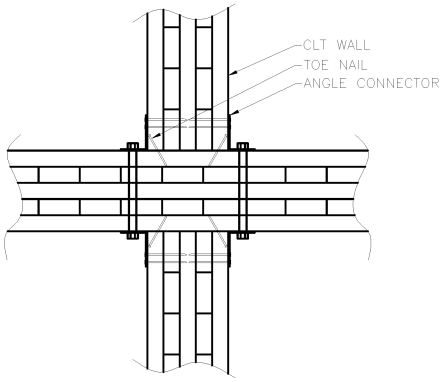


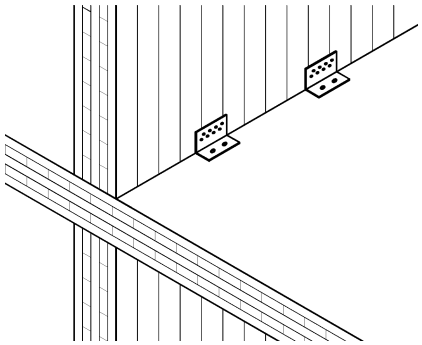


Interior wall to floor connection

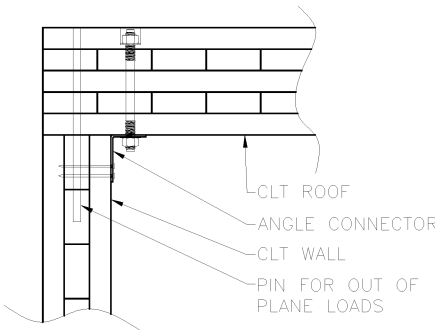




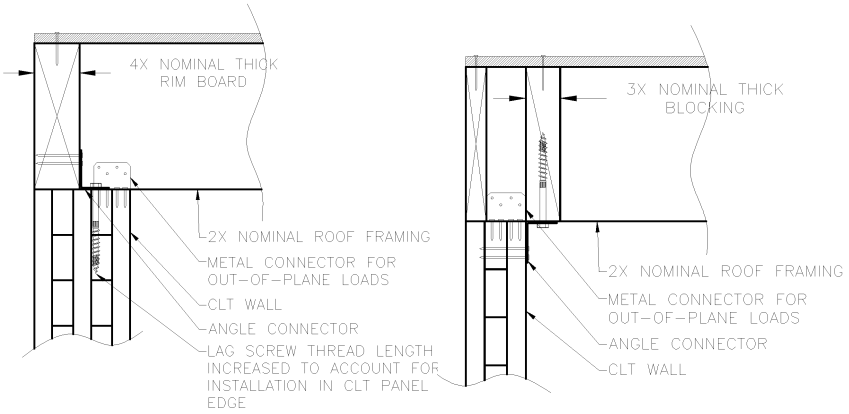




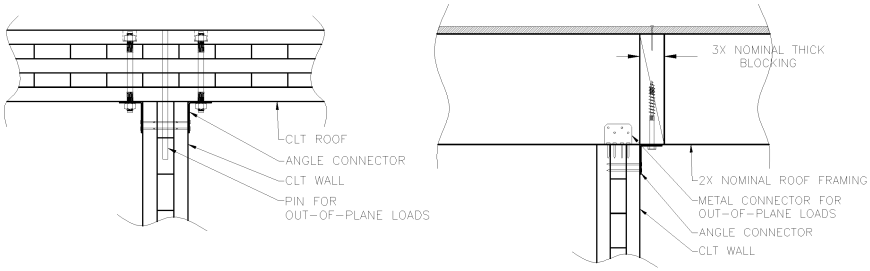
Exterior wall to roof



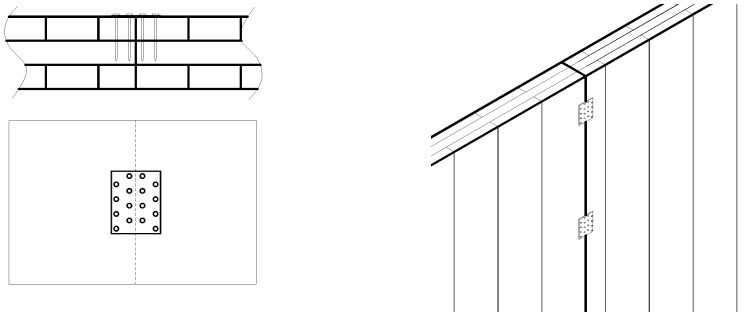
Exterior wall to roof



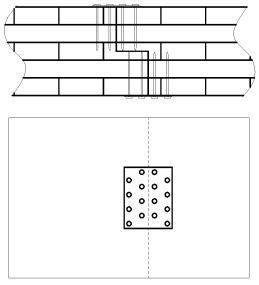
Interior wall to roof



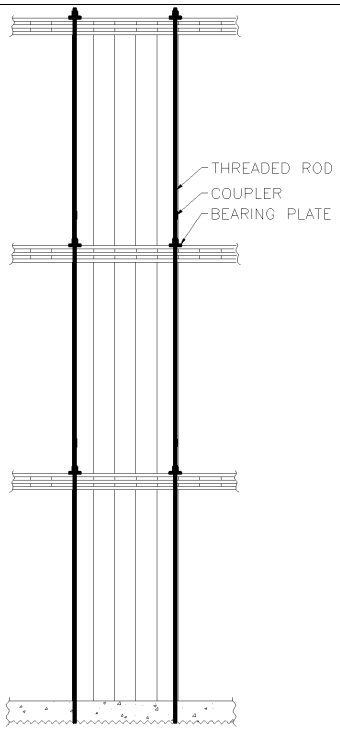
Wall panel to wall panel connection, square edge configuration (i.e., vertical edge connection)



Wall panel to wall panel connection, lap configuration (i.e., vertical edge connection)



Multistory overturning restraint



Appendix F—Cross-Laminated Timber Floor Reinforcement Option

Bearing capacity is based on unreinforced and reinforced options. The former refers to the case in which the cross-laminated timber (CLT) wall bears directly on the CLT floor, and compression perpendicular to the grain load and resistance factor design stress of 638 lb/in^2 (SPF) is used. Other values can be used based on wood species used for the CLT.

Where the reinforced option is used, bearing capacity is limited by parallel-to-the-grain capacity of the CLT wall instead of perpendicular-to-the-grain capacity of the CLT floor. Bearing stress values for the reinforced option, associated with the parallel-to-grain capacity of the CLT wall, are 968, 1,646, 1,905, and 1,895 lb/in^2 for CLT wall thicknesses of 4.125, 6.875, 9.625, and 12.375 in., respectively (Table F1). Calculations in accordance with the National Design Specification are provided in Figures F1, F2, F3, and F4.

Design options for controlling the length of the compression zone include increasing bearing area (such as by increasing wall thickness), use of CLT with increased compression perpendicular-to-grain stress, and use of reinforcement option (such as a designed bearing support, designed bearing plates and steel rods, designed squash blocking, and designed grout detail to bypass direct compression perpendicular-to-grain bearing on the CLT floor panel). An example reinforcement detail with steel parts designed for Omega forces is provided in Figure F5.

Table F1—Bearing stress values associated with the parallel-to-grain capacity of the CLT

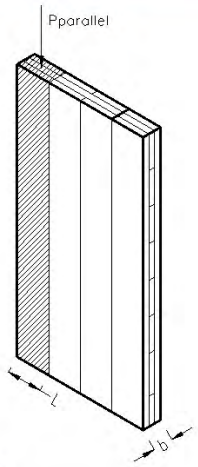
Panel thickness (in.)	Number of layers	Compression capacity, $F'_c A_{par}$ (lb/ft)[a]	$F'_c t_{par}$ (lb/in/ft)	A (in. by in.)	$F_c'(A_{par})/A$ (lb/in^2/ft)
4.125	3	47,922.9	3,993.0	12 by 4.125	968
6.875	5	135,789.1	11,316.3	12 by 6.875	1,646
9.625	7	219,991.0	18,335.6	12 by 9.625	1,905
12.375	9	281,335.8	23,450.6	12 by 12.375	1,895

[a]Calculated based on E1 grade CLT.

CLT COMPRESSION PARALLEL TO GRAIN

INPUT

b (Design width)=	1	ft
L (Wall height)=	10	ft



PANEL PROPERTIES

hi (Thickness of individual layers) =	1.375	in.
number of layers	3	
t (total thickness)=	4.125	in.

Major Strength axis (parallel to grain)

Fb,0 (Bending strength)=	875	psi
Eo (Modulus of elasticity)=	1.7E+06	psi
Ft,o (Tensile strength)=	450	psi
Fc,o (Compression strength)=	3888	psi
Fv,o (Shear strength)=	135	psi
Fs,o (Rolling shear strength)=	45	psi

Mino strength axis (perpendicular axis)

Fb,90 (Bending strength)=	500	psi
Eo(Modulus of elasticity)=	1.20E+06	psi
Fv,0 (Shear strength)=	135	psi
Fs,0 (Rolling shear strength)=	45	psi

Format Conversion	2.16	

ANALYSIS

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 =$$ **1.150E+08** lb-in2/ft

Layer	E psi	z (in.)	Ebh^3/12	Eaz^2	
1	1.70E+06	1.375	4.42E+06	5.30E+07	5.75E+07
2	4.00E+04	0	1.04E+05	0.00E+00	1.04E+05
3	1.70E+06	1.375	4.42E+06	5.30E+07	5.75E+07
4	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
5	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
6	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
7	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
8	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
9	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
					1.15E+08

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} =$$ **4.624E+05** lb/ft

a = 2.75

Layer	G	h/G/b or h/G/b/2
1	1.1E+05	5.4E-07
2	7500	1.5E-05
3	1.1E+05	5.4E-07
4	0	0.0E+00
5	0.0E+00	0.0E+00
6	0.0E+00	0.0E+00
7	0.0E+00	0.0E+00
8	0.0E+00	0.0E+00
9	0.0E+00	0.0E+00

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}} =$$ **9.553E+07** lb-in2

Ks= 11.8

EI app-min=0.5184*EI app= **7.41E+07** lb-in2/ft

Format Conversion 1.496

$$P_{cE} = \frac{\pi^2 EI'_{app-\min}}{l_e^2} =$$ **50780.07** lb/ft

Aparallel= 33 in2

Fc*Apar= 128304 lb/ft

ADJUSTMENT FACTORS

Load Duration Factor (CD)=	1	
Wet Service Factor (CM)=	1	
Temperature Factor (Ct)=	1	

Pc*= 128304 lb/ft

c= 0.9 for CLT

P_{cE}/Pc^*= 0.396

$$C_p = \frac{1 + \left(P_{cE}/P_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(P_{cE}/P_c^*\right)}{2c}\right]^2 - \frac{P_{cE}/P_c^*}{c}} =$$ **0.374**

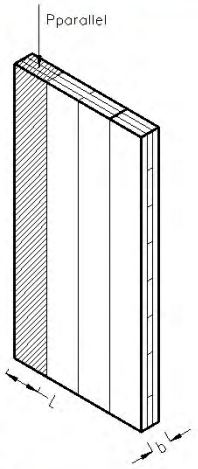
F'c*Apar= 47922.9 lb/ft

Figure F1. Parallel-to-the-grain capacity calculations for 4.125-in. CLT wall.

CLT COMPRESSION PARALLEL TO GRAIN

INPUT

b (Design width)=	1	ft
L (Wall height)=	10	ft



PANEL PROPERTIES

hi (Thickness of individual layers) =	1.375	in.
number of layers	5	
t (total thickness)=	6.875	in.

Major Strength axis (parallel to grain)

Fb,0 (Bending strength)=	875	psi
Eo (Modulus of elasticity)=	1.7E+06	psi
Ft,o (Tensile strength)=	450	psi
Fc,o (Compression strength)=	3888	psi
Fv,o (Shear strength)=	135	psi
Fs,o (Rolling shear strength)=	45	psi

Mino strength axis (perpendicular axis)

Fb,90 (Bending strength)=	500	psi
Eo(Modulus of elasticity)=	1.20E+06	psi
Fv,0 (Shear strength)=	135	psi
Fs,0 (Rolling shear strength)=	45	psi

Format Conversion	2.16

ANALYSIS

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \quad = \quad \textbf{4.402E+08} \text{ lb-in2/ft}$$

Layer	E psi	z (in.)	Ebh^3/12	Eaz^2	
1	1.70E+06	2.75	4.42E+06	2.12E+08	2.17E+08
2	4.00E+04	1.375	1.04E+05	1.25E+06	1.35E+06
3	1.70E+06	0	4.42E+06	0.00E+00	4.42E+06
4	4.00E+04	1.375	1.04E+05	1.25E+06	1.35E+06
5	1.70E+06	2.75	4.42E+06	2.12E+08	2.17E+08
6	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
7	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
8	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
9	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
					4.40E+08

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_i \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} \quad = \quad \textbf{9.247E+05} \text{ lb/ft}$$

a = 5.5

Layer	G	h/G/b or h/G/b/2
1	1.1E+05	5.4E-07
2	7500	1.5E-05
3	1.1E+05	1.1E-06
4	7500	1.5E-05
5	1.1E+05	5.4E-07
6	0.0E+00	0.0E+00
7	0.0E+00	0.0E+00
8	0.0E+00	0.0E+00
9	0.0E+00	0.0E+00

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}} \quad = \quad \textbf{3.167E+08} \text{ lb-in2}$$

Ks= 11.8

EI app-min=0.5184*EI app= **2.46E+08** lb-in2/ft

Format Conversion 1.496

$$P_{cE} = \frac{\pi^2 EI'_{app-\min}}{l_e^2} \quad = \quad \textbf{168327.81} \text{ lb/ft}$$

Aparallel= 49.5 in2

Fc*Apar= 192456 lb/ft

ADJUSTMENT FACTORS

Load Duration Factor (CD)=	1
Wet Service Factor (CM)=	1
Temperature Factor (Ct)=	1

Pc*= 192456 lb/ft

c= 0.9 for CLT

P_{cE}/Pc^*= 0.875

$$C_p = \frac{1 + \left(P_{cE}/P_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(P_{cE}/P_c^*\right)}{2c}\right]^2 - \frac{P_{cE}/P_c^*}{c}} \quad = \quad \textbf{0.706}$$

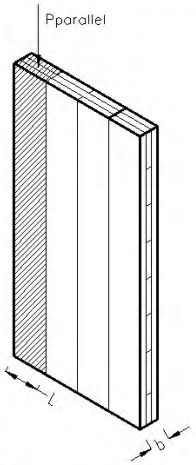
F'c*Apar= 135789.1 lb/ft

Figure F2. Parallel-to-the-grain capacity calculations for 6.875-in. CLT wall.

CLT COMPRESSION PARALLEL TO GRAIN

INPUT

b (Design width)=	1	ft
L (Wall height)=	10	ft



PANEL PROPERTIES

hi (Thickness of individual layers) =	1.375	in.
number of layers	7	
t (total thickness)=	9.625	in.

Major Strength axis (parallel to grain)

Fb,0 (Bending strength)=	875	psi
Eo (Modulus of elasticity)=	1.7E+06	psi
Ft,o (Tensile strength)=	450	psi
Fc,o (Compression strength)=	3888	psi
Fv,o (Shear strength)=	135	psi
Fs,o (Rolling shear strength)=	45	psi

Mino strength axis (perpendicular axis)

Fb,90 (Bending strength)=	500	psi
Eo(Modulus of elasticity)=	1.20E+06	psi
Fv,0 (Shear strength)=	135	psi
Fs,0 (Rolling shear strength)=	45	psi

Format Conversion 2.16

ANALYSIS

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \quad = \quad \textbf{1.089E+09}\ \text{lb-in2/ft}$$

Layer	E psi	z (in.)	Ebh^3/12	Eaz^2	
1	1.70E+06	4.125	4.42E+06	4.77E+08	4.82E+08
2	4.00E+04	2.75	1.04E+05	4.99E+06	5.10E+06
3	1.70E+06	1.375	4.42E+06	5.30E+07	5.75E+07
4	4.00E+04	0	1.04E+05	0.00E+00	1.04E+05
5	1.70E+06	1.375	4.42E+06	5.30E+07	5.75E+07
6	4.00E+04	2.75	1.04E+05	4.99E+06	5.10E+06
7	1.70E+06	4.125	4.42E+06	4.77E+08	4.82E+08
8	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
9	0.00E+00	0	0.00E+00	0.00E+00	0.00E+00
					1.09E+09

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} \quad = \quad \textbf{1.387E+06}\ \text{lb/ft}$$

a = 8.25

Layer	G	h/G/b or h/G/b/2
1	1.1E+05	5.4E-07
2	7500	1.5E-05
3	1.1E+05	1.1E-06
4	7500	1.5E-05
5	1.1E+05	1.1E-06
6	7.5E+03	1.5E-05
7	1.1E+05	5.4E-07
8	0.0E+00	0.0E+00
9	0.0E+00	0.0E+00

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}} \quad = \quad \textbf{6.625E+08}\ \text{lb-in2}$$

Ks= 11.8

EI app-min=0.5184*EI app= **5.14E+08** lb-in2/ft

Format Conversion 1.496

$$P_{cE} = \frac{\pi^2 EI'_{app-\min}}{l_e^2} \quad = \quad \textbf{352159.25}\ \text{lb/ft}$$

Aparallel= 66 in2

Fc*Apar= 256608 lb/ft

ADJUSTMENT FACTORS

Load Duration Factor (CD)=	1	
Wet Service Factor (CM)=	1	
Temperature Factor (Ct)=	1	

Pc*= 256608 lb/ft

c= 0.9 for CLT

P_{cE}/Pc^*= 1.372

$$C_p = \frac{1 + \left(P_{cE}/P_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(P_{cE}/P_c^*\right)}{2c}\right]^2 - \frac{P_{cE}/P_c^*}{c}} \quad = \quad \textbf{0.857}$$

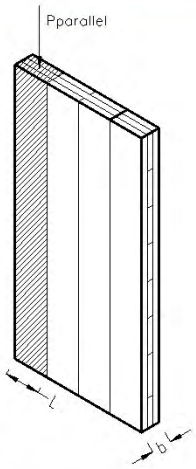
F'c*Apar= 219991.0 lb/ft

Figure F3. Parallel-to-the-grain capacity calculations for 9.625-in. CLT wall.

CLT COMPRESSION PARALLEL TO GRAIN

INPUT

b (Design width)=	1	ft
L (Wall height)=	10	ft



PANEL PROPERTIES

hi (Thickness of individual layers) =	1.375	in.
number of layers	9	
t (total thickness)=	12.375	in.

Major Strength axis (parallel to grain)

Fb,0 (Bending strength)=	875	psi
Eo (Modulus of elasticity)=	1.7E+06	psi
Ft,o (Tensile strength)=	450	psi
Fc,o (Compression strength)=	3888	psi
Fv,o (Shear strength)=	135	psi
Fs,o (Rolling shear strength)=	45	psi

Mino strength axis (perpendicular axis)

Fb,90 (Bending strength)=	500	psi
Eo(Modulus of elasticity)=	1.20E+06	psi
Fv,0 (Shear strength)=	135	psi
Fs,0 (Rolling shear strength)=	45	psi

Format Conversion 2.16

ANALYSIS

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 =$$ **1.304E+09** lb-in2/ft

Layer	E psi	z (in.)	Ebh^3/12	Eaz^2	
1	1.70E+06	5.5	4.42E+06	8.49E+08	8.53E+08
2	4.00E+04	4.125	1.04E+05	1.12E+07	1.13E+07
3	1.70E+06	2.75	4.42E+06	2.12E+08	2.17E+08
4	4.00E+04	1.375	1.04E+05	1.25E+06	1.35E+06
5	1.70E+06	0	4.42E+06	0.00E+00	4.42E+06
6	4.00E+04	1.375	1.04E+05	1.25E+06	1.35E+06
7	1.70E+06	2.75	4.42E+06	2.12E+08	2.17E+08
8	4.00E+04	4.125	1.04E+05	1.12E+07	1.13E+07
9	1.70E+06	5.5	4.42E+06	8.49E+08	8.53E+08
					1.30E+09

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} =$$ **2.439E+06** lb/ft

a = 11

Layer	G	h/G/b or h/G/b/2
1	1.1E+05	5.4E-07
2	7500	1.5E-05
3	1.1E+05	1.1E-06
4	7500	1.5E-05
5	1.1E+05	1.1E-06
6	7.5E+03	1.5E-05
7	1.1E+05	1.1E-06
8	7.5E+03	1.5E-05
9	1.1E+05	5.4E-07

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}} =$$ **9.070E+08** lb-in2

Ks= 11.8

EI app-min=0.5184*EI app= **7.03E+08** lb-in2/ft

Format Conversion 1.496

$$P_{cE} = \frac{\pi^2 EI'_{app-min}}{l_e^2} =$$ **482100.75** lb/ft

Aparallel= 82.5 in2

Fc*Apar= 320760 lb/ft

ADJUSTMENT FACTORS

Load Duration Factor (CD)=	1
Wet Service Factor (CM)=	1
Temperature Factor (Ct)=	1

Pc*= 320760 lb/ft

c= 0.9 for CLT

P_{cE}/Pc^*= 1.503

$$C_p = \frac{1 + \left(P_{cE}/P_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(P_{cE}/P_c^*\right)}{2c}\right]^2 - \frac{P_{cE}/P_c^*}{c}} =$$ **0.877**

F'c*Apar= 281335.8 lb/ft

Figure F4. Parallel-to-the-grain capacity calculations for 12.375-in. CLT wall.

CLT PERP TO GRAIN REINFORCEMENT DESIGN

Steel bearing plate and rods are designed based on the AISC 360 provisions. In this case bearing plate is assumed to be subjected to uniform load and is designed as simply supported span between the rod supports. Rods are designed as pin end supported compression members based on Chapter D of AISC 360.

INPUT

Load, W=	138.7	kip
Omega, Ω=	3.0	
b (Design width)=	12.375	in.
L (Design length)=	5.88	in.
t_floor (floor thickness)=	6.875	in.

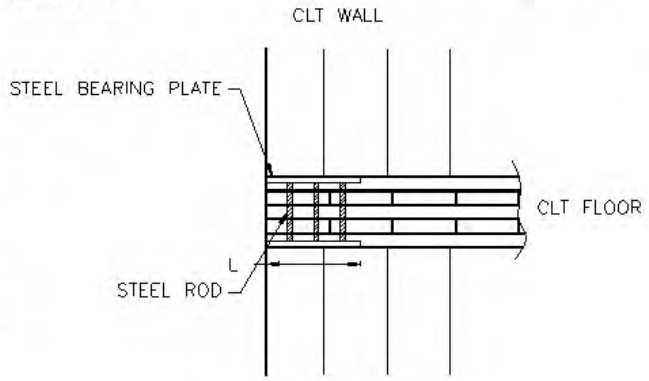
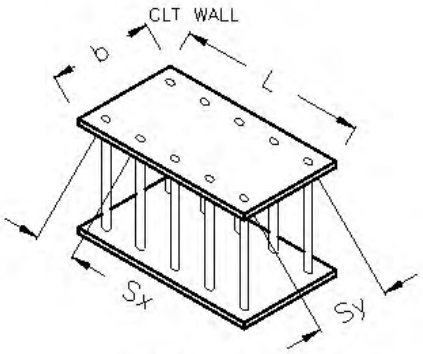


PLATE PROPERTIES

Fy_p (Plate yield strength)	50	ksi
t (Plate thickness) =	0.875	in.
E (Modulus of Elasticity)	29000	ksi

ROD PROPERTIES

Rod diamter,d	1	in.
Fy_rod (Rod yield strength)	50	ksi
E (Modulus of Elasticity)	29000	ksi
sx (spacing along length of the wall)	2.5	in.
sy (spacing along thickness of the wall)	2.5	in.
nx (rows of rods along length of the wall)	3	
ny (rows of rods along thickness of the wall)	5	



ANALYSIS

ROD DESIGN

K (Effective legnth factor)	1	
L	6.875	in.
Lc (Effective length of the member)	6.875	in.

$$\frac{L_c}{r} = 13.8 < 4.71\sqrt{\frac{E}{F_y}} = 113.43$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = 49.31 \text{ ksi}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} = 1513.88 \text{ ksi}$$

$$P_n = F_{cr}A_g = 580.96 \text{ kips}$$

$$\phi = 0.9$$

$$\phi P_n = 522.87 \text{ kips} > 416.0 \text{ kips}$$

PLATE DESIGN

w_x	=	14.29	kip/in
w_y	=	14.29	kip/in
M_x	=	11.17	kip-in.
M_y	=	11.17	kip-in.
M_u	=	11.17	kip-in.
S	=	0.319	in^3
M_n	=	15.95	kip-in.
ϕ	=	0.9	
ϕM_n	=	14.36	kip-in. > 11.17 kip-in.

Figure F5. Example compression zone reinforcement detail for Story 1 of the example archetype.

Appendix G—CUREE Parameters used in Modeling

Table G1—Hysteretic parameters used for nonlinear analysis: 2.5-ft panels with two connectors per panel

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
2.5	1	A3	2	S	2,250	7,250	650	0.075	–0.125	0.825	0.05	8.75	0.75	1.05
2.5	1	A3	2	D	4,500	14,500	1,300	0.075	–0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	2	S	7,500	14,500	1,600	0.075	–0.125	0.825	0.05	6.5	0.75	1.05
2.5	2	A3	2	D	15,000	29,000	3,200	0.075	–0.125	0.825	0.05	6.5	0.75	1.05
2.5	3	A3	2	S	13,250	18,250	2,550	0.075	–0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A3	2	D	26,500	36,500	5,100	0.075	–0.125	0.825	0.05	6.125	0.75	1.05
2.5	4	A3	2	S	19,000	22,000	3,500	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	4	A3	2	D	38,000	44,000	7,000	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	2	S	24,750	25,750	4,450	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	2	D	49,500	51,500	8,900	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	2	S	30,500	29,500	5,400	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	2	D	61,000	59,000	10,800	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	2	S	42,000	37,000	7,300	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	2	D	84,000	74,000	14,600	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	2	S	47,750	40,750	8,250	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	2	D	95,500	81,500	16,500	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	2	S	53,500	44,500	9,200	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	2	D	107,000	89,000	18,400	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	2	S	99,500	74,500	16,800	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	2	D	199,000	149,000	33,600	0.075	–0.125	0.825	0.05	5.75	0.75	1.05

Table G2—Hysteretic parameters used for nonlinear analysis: 2.5-ft panels with three connectors per panel

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
2.5	1	A3	3	S	3,375	10,875	975	0.075	–0.125	0.825	0.05	8.75	0.75	1.05
2.5	1	A3	3	D	6,750	21,750	1,950	0.075	–0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	3	S	11,250	21,750	2,400	0.075	–0.125	0.825	0.05	6.5	0.75	1.05
2.5	2	A3	3	D	22,500	43,500	4,800	0.075	–0.125	0.825	0.05	6.5	0.75	1.05
2.5	3	A3	3	S	19,875	27,375	3,825	0.075	–0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A3	3	D	39,750	54,750	7,650	0.075	–0.125	0.825	0.05	6.125	0.75	1.05
2.5	4	A3	3	S	28,500	33,000	5,250	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	4	A3	3	D	57,000	66,000	10,500	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	3	S	37,125	38,625	6,675	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	3	D	74,250	77,250	13,350	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	3	S	45,750	44,250	8,100	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	3	D	91,500	88,500	16,200	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	3	S	63,000	55,500	10,950	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	3	D	126,000	111,000	21,900	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	3	S	71,625	61,125	12,375	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	3	D	143,250	122,250	24,750	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	3	S	80,250	66,750	13,800	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	3	D	160,500	133,500	27,600	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	3	S	149,250	111,750	25,200	0.075	–0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	3	D	298,500	223,500	50,400	0.075	–0.125	0.825	0.05	5.75	0.75	1.05

Table G3—Hysteretic parameters used for nonlinear analysis: 5-ft single panel

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	1	A3	2	S	5,000	8,760	1,500	0.05	–0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	S	6,000	16,700	1,500	0.1	–0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	S	8,000	17,670	2,000	0.125	–0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	S	10,000	21,460	2,750	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	6	S	12,000	25,752	3,300	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	7	S	14,000	30,044	3,850	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	8	S	16,000	34,336	3,850	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	9	S	18,000	38,628	4,400	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	2	D	10,000	17,520	3,000	0.05	–0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	D	12,000	33,400	3,000	0.1	–0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	D	16,000	35,340	4,000	0.125	–0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	D	20,000	42,920	5,500	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	6	D	24,000	51,504	6,600	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	7	D	28,000	60,088	7,700	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	8	D	32,000	68,672	7,700	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	9	D	36,000	77,256	8,800	0.05	–0.15	1.05	0.05	3.75	0.75	1.02

Table G4—Hysteretic parameters used for nonlinear analysis: two 5-ft panels

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	2	A3	2	S	16,666.7	17,520.0	3,692.3	0.05	–0.15	0.80	0.05	2.41	0.75	1.02
5	2	A3	3	S	20,000.0	33,400.0	3,692.3	0.10	–0.40	1.05	0.05	2.97	0.60	1.02
5	2	A3	4	S	26,666.7	35,340.0	4,923.1	0.13	–0.20	1.05	0.05	2.60	0.60	1.02
5	2	A3	5	S	33,333.3	42,920.0	6,769.2	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	6	S	40,000.0	51,504.0	8,123.1	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	7	S	46,666.7	60,088.0	9,476.9	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	8	S	53,333.3	68,672.0	9,476.9	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	2	D	33,333.3	35,040.0	7,384.6	0.05	–0.15	0.80	0.05	2.41	0.75	1.02
5	2	A3	3	D	40,000.0	66,800.0	7,384.6	0.10	–0.40	1.05	0.05	2.97	0.60	1.02
5	2	A3	4	D	53,333.3	70,680.0	9,846.2	0.13	–0.20	1.05	0.05	2.60	0.60	1.02
5	2	A3	5	D	66,666.7	85,840.0	13,538.5	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	6	D	80,000.0	103,008.0	16,246.2	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	7	D	93,333.3	120,176.0	18,953.8	0.05	–0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	8	D	106,666.7	137,344.0	18,953.8	0.05	–0.15	1.05	0.05	2.79	0.75	1.02

Table G5—Hysteretic parameters used for nonlinear analysis: three 5-ft panels

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	3	A3	2	S	29,444.4	22,051.0	5,884.6	0.05	–0.15	0.80	0.05	2.3	0.8	1.0
5	3	A3	3	S	35,333.3	42,037.9	5,884.6	0.10	–0.40	1.05	0.05	2.8	0.6	1.0
5	3	A3	4	S	47,111.1	44,479.7	7,846.2	0.13	–0.20	1.05	0.05	2.5	0.6	1.0
5	3	A3	5	S	58,888.9	54,020.0	10,788.5	0.05	–0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	6	S	70,666.7	64,824.0	12,946.2	0.05	–0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	7	S	82,444.4	75,628.0	15,103.8	0.05	–0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	8	S	94,222.2	86,432.0	15,103.8	0.05	–0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	2	D	58,888.9	44,102.1	11,769.2	0.1	–0.2	0.8	0.1	2.3	0.8	1.0
5	3	A3	3	D	70,666.7	84,075.9	11,769.2	0.10	–0.40	1.05	0.05	2.80	0.6	1.0
5	3	A3	4	D	94,222.2	88,959.3	15,692.3	0.13	–0.20	1.05	0.05	2.45	0.6	1.0
5	3	A3	5	D	117,777.8	108,040.0	21,576.9	0.05	–0.15	1.05	0.05	2.63	0.8	1.0
5	3	A3	6	D	141,333.33	129,648	25,892.31	0.05	–0.15	1.05	0.05	2.63	0.75	1.02
5	3	A3	7	D	164,888.89	151,256	30,207.69	0.05	–0.15	1.05	0.05	2.63	0.75	1.02
5	3	A3	8	D	188,444.44	172,864	30,207.69	0.05	–0.15	1.05	0.05	2.63	0.75	1.02

Table G6—Hysteretic parameters used for nonlinear analysis: four 5-ft panels

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	4	A3	2	S	42,222.2	26,582.1	8,076.9	0.05	–0.15	0.80	0.05	2.14	0.75	1.02
5	4	A3	3	S	50,666.7	50,675.9	8,076.9	0.10	–0.40	1.05	0.05	2.63	0.60	1.02
5	4	A3	4	S	67,555.6	53,619.3	10,769.2	0.13	–0.20	1.05	0.05	2.30	0.60	1.02
5	4	A3	5	S	84,444.4	65,120.0	14,807.7	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	6	S	101,333.33	78,144	17,769.23	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	7	S	118,222.22	91,168	20,730.77	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	8	S	135,111.11	104,192	20,730.77	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	2	D	84,444.4	53,164.1	16,153.8	0.05	-0.15	0.80	0.05	2.14	0.75	1.02
5	4	A3	3	D	101,333.3	101,351.7	16,153.8	0.10	-0.40	1.05	0.05	2.63	0.60	1.02
5	4	A3	4	D	135,111.1	107,238.6	21,538.5	0.13	-0.20	1.05	0.05	2.30	0.60	1.02
5	4	A3	5	D	168,888.9	130,240.0	29,615.4	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	6	D	202,666.67	156,288	35,538.46	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	7	D	236,444.44	182,336	41,461.54	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	8	D	270,222.22	208,384	41,461.54	0.05	-0.15	1.05	0.05	2.46	0.75	1.02

Table G7—Hysteretic parameters used for nonlinear analysis: nine 5-ft panels

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	9	A3	2	S	106,111.1	49,237.2	19,038.5	0.05	–0.15	0.80	0.05	2.14	0.75	1.02
5	9	A3	3	S	127,333.3	93,865.5	19,038.5	0.10	–0.40	1.05	0.05	2.63	0.60	1.02
5	9	A3	4	S	169,777.8	99,317.6	25,384.6	0.13	–0.20	1.05	0.05	2.30	0.60	1.02
5	9	A3	5	S	212,222.2	120,620.0	34,903.8	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	6	S	254,666.67	144,744	41,884.62	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	7	S	297,111.11	168,868	48,865.38	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	8	S	339,555.56	192,992	48,865.38	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	2	D	212,222.2	98,474.5	38,076.9	0.05	–0.15	0.80	0.05	2.14	0.75	1.02
5	9	A3	3	D	254,666.7	187,731.0	38,076.9	0.10	–0.40	1.05	0.05	2.63	0.60	1.02
5	9	A3	4	D	339,555.6	198,635.2	50,769.2	0.13	–0.20	1.05	0.05	2.30	0.60	1.02
5	9	A3	5	D	424,444.4	241,240.0	69,807.7	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	6	D	509,333.33	289,488	83,769.23	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	7	D	594,222.22	337,736	97,730.77	0.05	–0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	8	D	679,111.11	385,984	97,730.77	0.05	–0.15	1.05	0.05	2.46	0.75	1.02

Appendix H—Example Application

Description of the Index Buildings

The structure presented here is a multifamily residential structure with a rectangular plan of 40 by 60 ft. The floor plan is identical for each story and is shown in Figure H1. Each floor has four relatively small apartments and elevator and stair access. Each story has a clear height of 10 ft, and the method of construction is platform, whereby floor panels bear on and are supported by the vertical cross-laminated timber (CLT) panels below. Figure H2 illustrates assignment of the shear walls used as part the lateral force resisting system.

Gravity Loads and Seismic Weight

Typical composition of the exterior wall, interior wall, floor and roof materials, and the corresponding unit weights, presented earlier in Appendix A, were used to calculate the seismic weight of the structure. These assigned layers are for the purpose of this example and this project only and are not verified for fire and sound performance. The seismic weight summary for the six-story Index Building 4 is provided in Table H1, and detailed calculations and the floor plan are provided in Table H2 and Figure H3, respectively.

Archetypes

The design space was divided into various performance groups based on the factors that significantly affect seismic behavior. The list of archetypes extracted from shear wall lines 3, B, and E of Index Building 4 that fall under different performance groups is given in Table H3. The naming system is as follows:

Index Building_Extracted Wall Name_Number of Stories_Low_Basic Configuration_High or Mixed Aspect Ratio_Low or High Gravity_Seismic Design Category D_{max} or D_{min}_Short Period or Long Period

The design example archetype description is 4_3_6_1_MR_HG_DX_LP. This indicates that it is Wall 3 of Index Building 4 with six stories, fits basic configuration of 2.5-20 ft, designed with a combination of high and low aspect ratio panels in the archetype, low gravity, seismic design category (SDC) D_{max}, and long period archetype. The tributary area and the extracted archetype are shown in Figures H4 and H5, respectively.

Seismic load calculations for the wall line are provided in Table H4, and the tributary area is shown in Figure H6. The highlighted values in Table H4 were the values used in the seismic load calculations and subsequent design of the archetypes (Table H5). Two shear walls are assumed to be along the wall line, and each one is intended to carry half of the seismic load.

Design

With the forces computed, as shown in Table H6, archetype design was performed based on the design methodology as explained in Chapter 5 of this document. Archetypes are to be designed for shear forces as well as the associated overturning moment (Table H7).

The shear wall shown in Figure H5 consists of a single 5- by 10-ft and (3) 2.5- by 10-ft multipanel configuration. The designed shear wall is shown in Figure H7, and the following demonstrates design calculations for the 5- by 10-ft segment.

- CLT panel size: 5 by 10 ft (CLT panel aspect ratio = h/b_s = 2:1) per Section 5.2-2a.
- CLT shear wall at each level: 5 by 10 ft (CLT shear wall aspect ratio=h/b=2:1).
- Connector type: Type A for top and bottom of the shear wall and Type E for the interpanel connector that is equivalent to a Type A connector.
- Number of connectors per panel was chosen to satisfy spacing requirements specified in Section 5.3.2.
- Nominal unit shear: The unit shear capacity is calculated for single- and double-sided configuration based on Section 5.5.
- Design unit shear (load and resistance factor design (LRFD)): Section 5.6. ϕ_D = 0.50

Looking at the following table based on the design methodology, we have

Nominal unit shear capacity of CLT special shear walls, lb/ft

Base and top of wall connection	Vertical joint connection	Minimum panel thickness, inch	Nominal unit shear capacity, v_s, lb/ft
Section 5.3.2	Section 5.3.3	3.5 (Section 5.3.1)	$v_s = n\left(\frac{2{,}605}{b_s}\right)$ (Eq. 5.2)

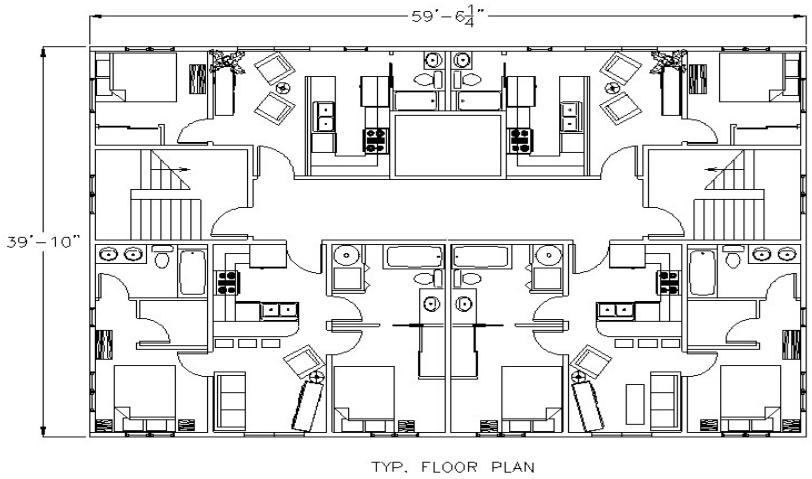


Figure H1. Typical floor plan.

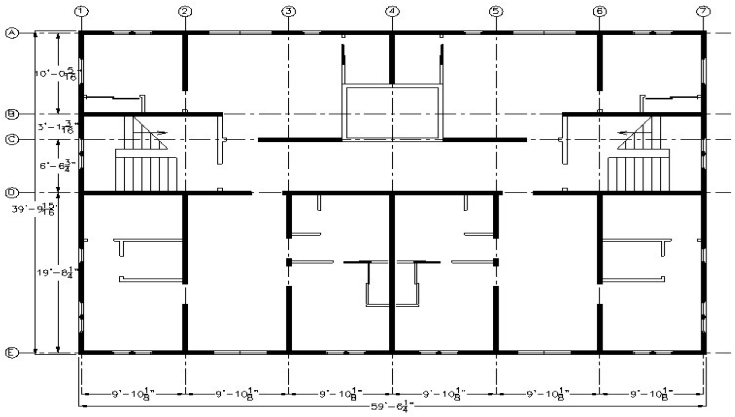


Figure H2. Assigned shear wall lines.

Table H1—Seismic weight summary for Index Building 4

Level	Height (ft)	W_{Level}, light (kips)	W_{Level}, heavy (kips)
Roof		125.1	125.1
	10		
5		180.7	228.4
	10		
4		180.7	228.4
	10		
3		180.7	228.4
	10		
2		180.7	228.4
	10		
1		183.1	230.8
	10		
Ground			

Table H2—Six-story Index Building 4 detailed seismic weight calculation

Level	Story	Height (ft)	A_{floor}	$L_{Extwall}$	$A_{ExtWall}$-openings	$L_{Intwall}$	$A_{IntWall}$-openings	Light W_{floor} (lb)	Light $W_{ExtWalls}$ (lb)	Light $W_{IntWalls}$ (lb)	Light ΣW_{Level} (lb)	Heavy W_{floor} (lb)	Heavy $W_{ExtWalls}$ (lb)	Heavy $W_{IntWalls}$ (lb)	Heavy ΣW_{Level} (lb)
Roof			2,371					69,391.27			**125,107**	69,391.27			**125,107**
	6	10		196.83	1,636.33	234.917	2,169		55,179	56,254			55,179	56,254	
5			2,371					69,312.23			**185,486**	116,999			**228,431**
	5	10		196.83	1,636.33	234.917	2,169		55,179	56,254			55,179	56,254	
4			2,371					69,312.23			**185,486**	116,999			**228,431**
	4	10		196.83	1,636.33	234.917	2,169		55,179	56,254			55,179	56,254	
3			2,371					69,312.23			**185,486**	116,999			**228,431**
	3	10		196.83	1,636.33	234.917	2,169		55,179	56,254			55,179	56,254	
2			2,371					69,312.23			**185,486**	116,999			**228,431**
	2	10		196.83	1,636.33	234.917	2,169		55,179	56,254			55,179	56,254	
1			2,371					69,312.23			**187,820**	116,999			**230,765**
	1	10		196.83	1,636.33	234.917	2,349		55,179	60,922			55,179	60,922	
Ground			0												

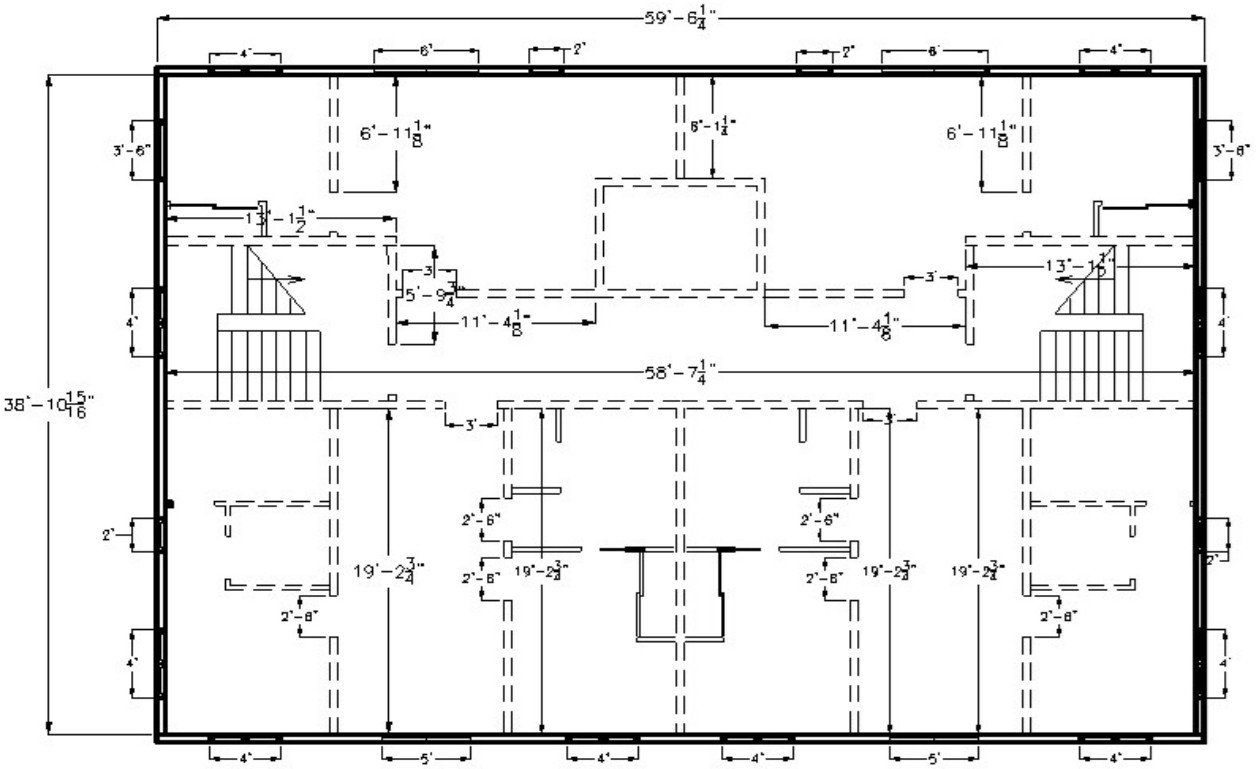


Figure H3. Index Building 4 interior and exterior wall dimensions.

Table H3—Extracted archetypes from Index Building 4

Group No.		Basic configuration	Design load level: Gravity	Design load level: Seismic	Archetype description	Archetype No.
PG-2	2.5- to 20-ft wall	Low aspect ratio panels	High	SDC D_{max}	4_3_6_1_LR_HG_DX_LP	4
PG-6			Low		4_3_6_1_LR_LG_DX_LP	16
PG-10		High aspect ratio panels	High		4_3_6_1_HR_HG_DX_LP	28
PG-14			Low		4_3_6_1_HR_LG_DX_LP	40
PG-18		Mixed aspect ratio	High		4_3_6_1_MR_HG_DX_LP	52
					4_B_6_1_MR_HG_DX_LP	54
PG-22			Low		4_3_6_1_MR_LG_DX_LP	64
					4_B_6_1_MR_LG_DX_LP	66
PG-17			High		4_3_4_1_MR_HG_DX_SP	50
PG-21			Low		4_3_4_1_MR_LG_DX_SP	62
PG-26	20- to 60-ft wall	Low aspect ratio panels	High		4_E_6_2_LR_HG_DX_LP	76
PG-30			Low		4_E_6_2_LR_LG_DX_LP	88
PG-34		High aspect ratio panels	High		4_E_6_2_HR_HG_DX_LP	100
PG-38			Low		4_E_6_2_HR_LG_DX_LP	112
PG-42		Mixed aspect ratio	High		4_E_6_2_MR_HG_DX_LP	124
PG-46			Low		4_E_6_2_MR_LG_DX_LP	136

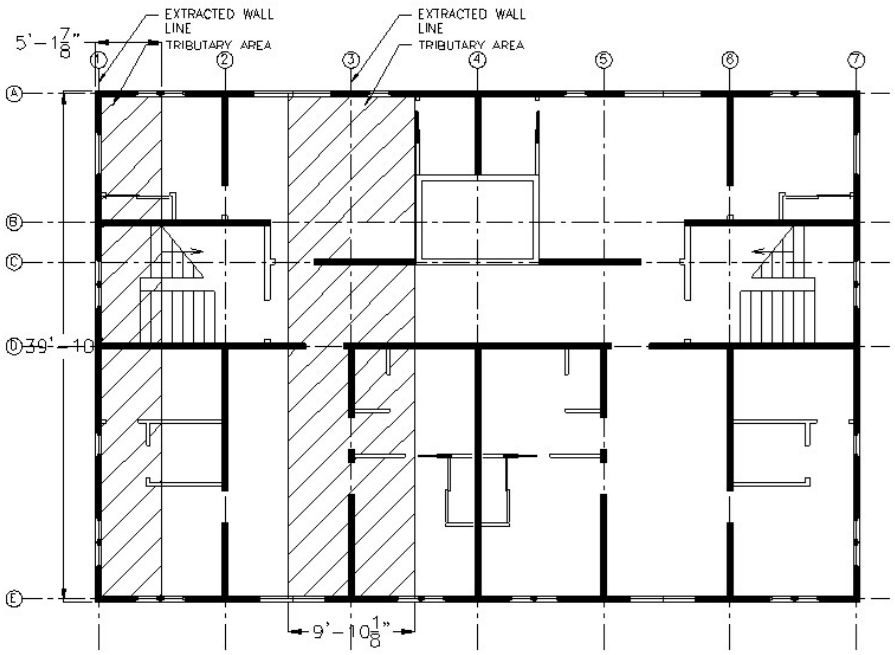


Figure H4. Tributary area.

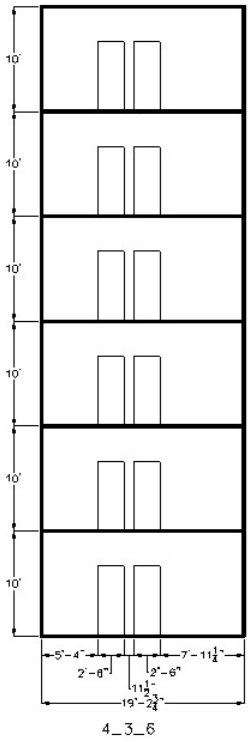


Figure H5. Shear wall.

Equivalent Lateral Force Procedure:

INPUT DATA

Total Height	h_n=	**60.0**	ft				
Total Weight	W=	**1,270**	k				
Seismic Design Category		**Dmax**					
Importance factor (ASCE 11.5.1)	I =	**1**	(IBC Tab.1604.5)				
	S_S =	**1.500**	%g , S_{ms} =	1.500	g , F_a =	**1.000**	
	S_1 =	**0.600**	%g , S_{m1} =	0.900	g , F_v =	**1.500**	
	S_{DS} =	**1.000**	g ,				
	S_{D1} =	**0.600**	g				
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)				
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**					
The coefficient(ASCE Tab. 12.2.1)	R =	**3**					
	x =	**0.75**	, (ASCE Tab 12.8-2)				
	$T_a = C_t(h_n)^x$ =	0.43	Sec, (ASCE 12.8.2.1)				
	Cu=	**1.40**					
	T=Cu*Ta=	0.6036					
	Ts=	0.6					
	Cs=	0.3313					
	k =	**1.05**	, (ASCE 12.8.3, pg 130)				
	$\Sigma w_x h^k$ =	50,434					

DESIGN SUMMARY

Total base shear

V = 420.62

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft
6		60.0	**125**	9,280	0.184	77.4		
	10.00						77.4	
5		50.0	**228**	13,986	0.277	116.6		774
	10.00						194.0	
4		40.0	**228**	11,060	0.219	92.2		2,714
	10.00						286.3	
3		30.0	**228**	8,173	0.162	68.2		5,577
	10.00						354.4	
2		20.0	**228**	5,335	0.106	44.5		9,122
	10.00						398.9	
1		10.0	**231**	2,600	0.052	21.7		13,111
	10.00						420.6	
		0.0						17,317

Table H4—Tributary load calculation

	Story shear (kip)	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Cumulative shear load (kip)
			Heavy weight		
Story 6	58.05	1	205.0	0.086	6.69
		2	392.0	0.165	12.80
		3	392.0	0.165	12.80
		4	392.0	0.165	12.80
		5	392.0	0.165	12.80
		6	392.0	0.165	12.80
		7	205.0	0.086	6.69
Story 5	145.53	1	205.0	0.086	16.78
		2	392.0	0.165	32.09
		3	392.0	0.165	32.09
		4	392.0	0.165	32.09
		5	392.0	0.165	32.09
		6	392.0	0.165	32.09
		7	205.0	0.086	16.78
Story 4	214.71	1	205.0	0.086	24.76
		2	392.0	0.165	47.35
		3	392.0	0.165	47.35
		4	392.0	0.165	47.35
		5	392.0	0.165	47.35
		6	392.0	0.165	47.35
		7	205.0	0.086	24.76
Story 3	265.83	1	205.0	0.086	30.66
		2	392.0	0.165	58.62
		3	392.0	0.165	58.62
		4	392.0	0.165	58.62
		5	392.0	0.165	58.62
		6	392.0	0.165	58.62
		7	205.0	0.086	30.66
Story 2	299.20	1	205.0	0.086	34.51
		2	392.0	0.165	65.98
		3	392.0	0.165	65.98
		4	392.0	0.165	65.98
		5	392.0	0.165	65.98
		6	392.0	0.165	65.98
		7	205.0	0.086	34.51
Story 1	315.47	1	205.0	0.086	36.38
		2	392.0	0.165	69.57
		3	392.0	0.165	69.57
		4	392.0	0.165	69.57
		5	392.0	0.165	69.57
		6	392.0	0.165	69.57
		7	205.0	0.086	36.38

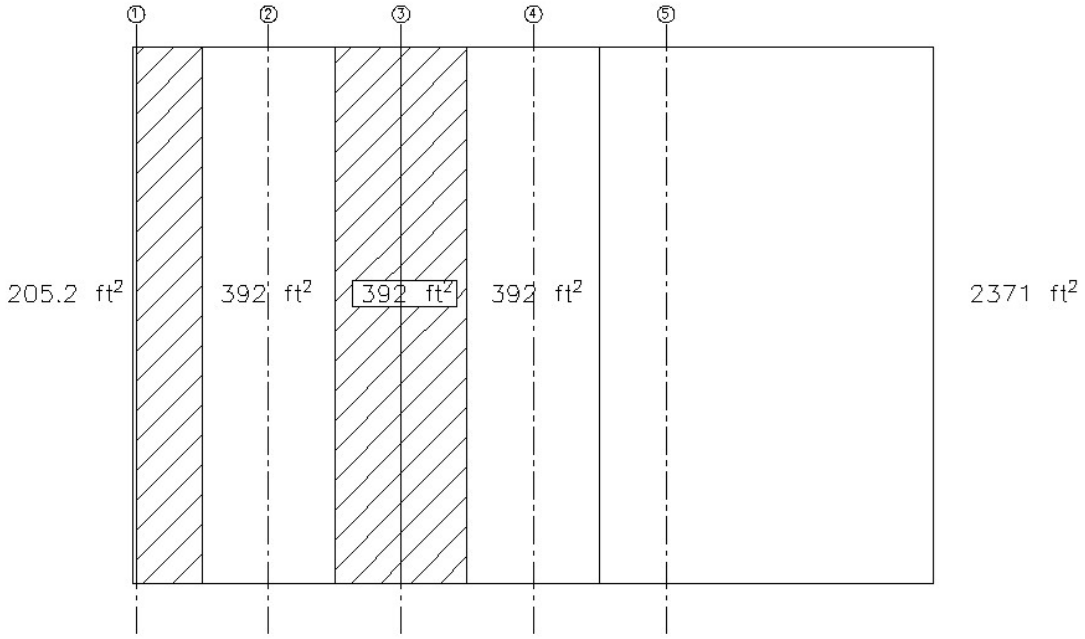


Figure H6. Index Building 4 seismic tributary area for north–south direction walls.

Table H5—Archetype tributary seismic load

Story	Seismic load (kips)[a]
6	6.4
5	16.0
4	23.7
3	29.3
2	33.0
1	34.8

[a]These values were obtained by dividing the highlighted values in Table H4 by two. This was done because there was not adequate capacity using the smallest connector spacing with Connector Type A. Instead of eliminating the archetypes or revising the index building design, double shear walls were assumed along shear wall line 3.

Table H6—Archetype shear design

Story	No. of panels	Panel length (ft)	Connector type	Number of connectors/ side/panel	S/D	Shear capacity (lb/ft)[a]	Stiffness (kip/in.)[a]	Applied load (lb/ft)[a]	Shear strength provided (kip)	Story shear (kip)/archetype	Ratio of provided shear to story shear
6	1	5	A	2	S	521.0	0.80	436.7	10.42	6.40	1.63
5	1	5	A	3	D	1,563.0	1.99	1,523.3	19.54	16.05	1.22
4	1	5	A	5	D	2,605.0	3.29	2,505.5	28.66	23.68	1.21
3	1	5	A	5	D	2,605.0	3.16	2,536.0	36.47	29.31	1.24
2	1	5	A	5	D	2,605.0	3.15	2,553.0	36.47	32.99	1.11
1	1	5	A	6	D	3,126.0	3.41	2,881.7	39.08	34.79	1.12
6	3	2.5	A	2	S	1,042.0	1.55	562.3			
5	3	2.5	A	3	S	1,563.0	2.20	1,124.1			
4	3	2.5	A	2	D	2,084.0	2.93	1,486.4			
3	3	2.5	A	3	D	3,126.0	4.14	2,217.6			
2	3	2.5	A	3	D	3,126.0	4.99	2,696.9			
1	3	2.5	A	3	D	3,126.0	4.83	2,716.9			

[a]Shear load was distributed based on relative stiffness of the shear walls in the shear wall line. Applied loads to individual shear walls were calculated based on an iterative procedure to ensure deflection compatibility for the shear walls along the wall line.

Table H7—Shear design calculations for CLT panels used in the archetype

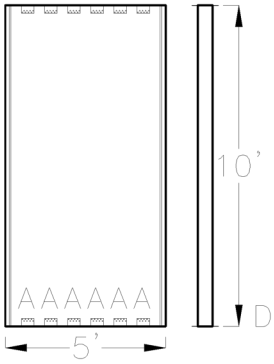
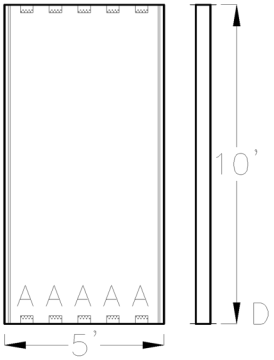
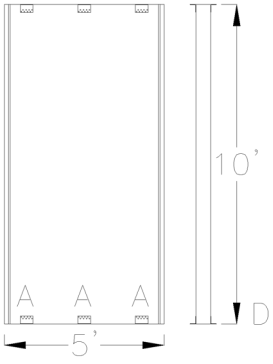
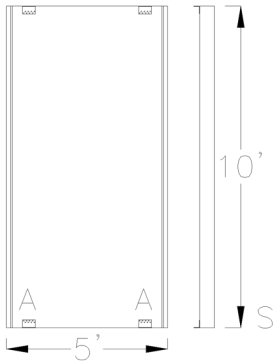
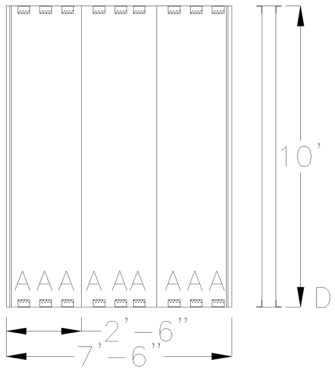
Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
5	A3	D	6	= 2 × 6 × 2,605/5 = 6,252	3,126
					
Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
5	A3	D	5	= 2 × 5 × 2,605/5 = 5,210	2,605
					
Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
5	A3	D	3	= 2 × 3 × 2,605/5 = 3,126	1,563
					

Table H7—Shear design calculations for CLT panels used in the archetype—continued

Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
5	A3	S	2	= 1 × 2 × 2,605/5 = 1,042	521



Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
2.5	A3	D	2	= 2 × 3 × 3 × 2,605/7.5 = 6,252	3,126



Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
2.5	A3	S	2	= 1 × 3 × 3 × 2,605/7.5 = 3,126	1,563

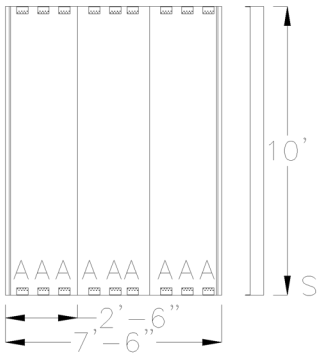
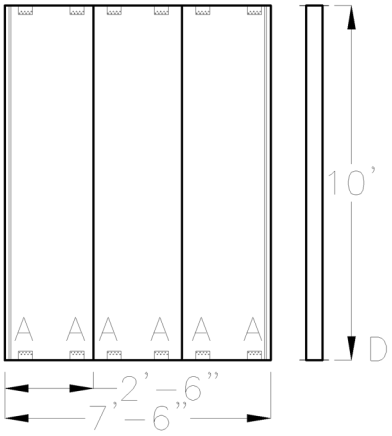
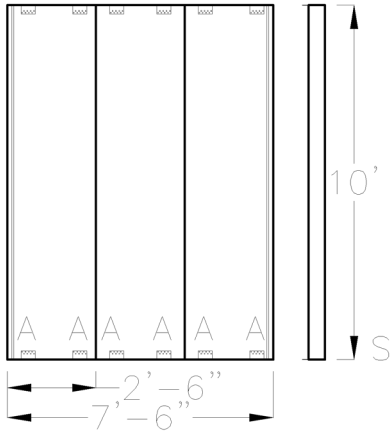


Table H7—Shear design calculations for CLT panels used in the archetype—continued

Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
2.5	A3	D	2	= 2 × 3 × 2 × 2,605/7.5 = 4,168	2,084



Panel length (ft)	Connector type	S/D	Number of connectors on one side per panel	Nominal unit shear capacity (lb/ft)	Design unit shear capacity (lb/ft)
2.5	A3	S	2	= 1 × 3 × 2 × 2,605/7.5 = 2,084	1,042



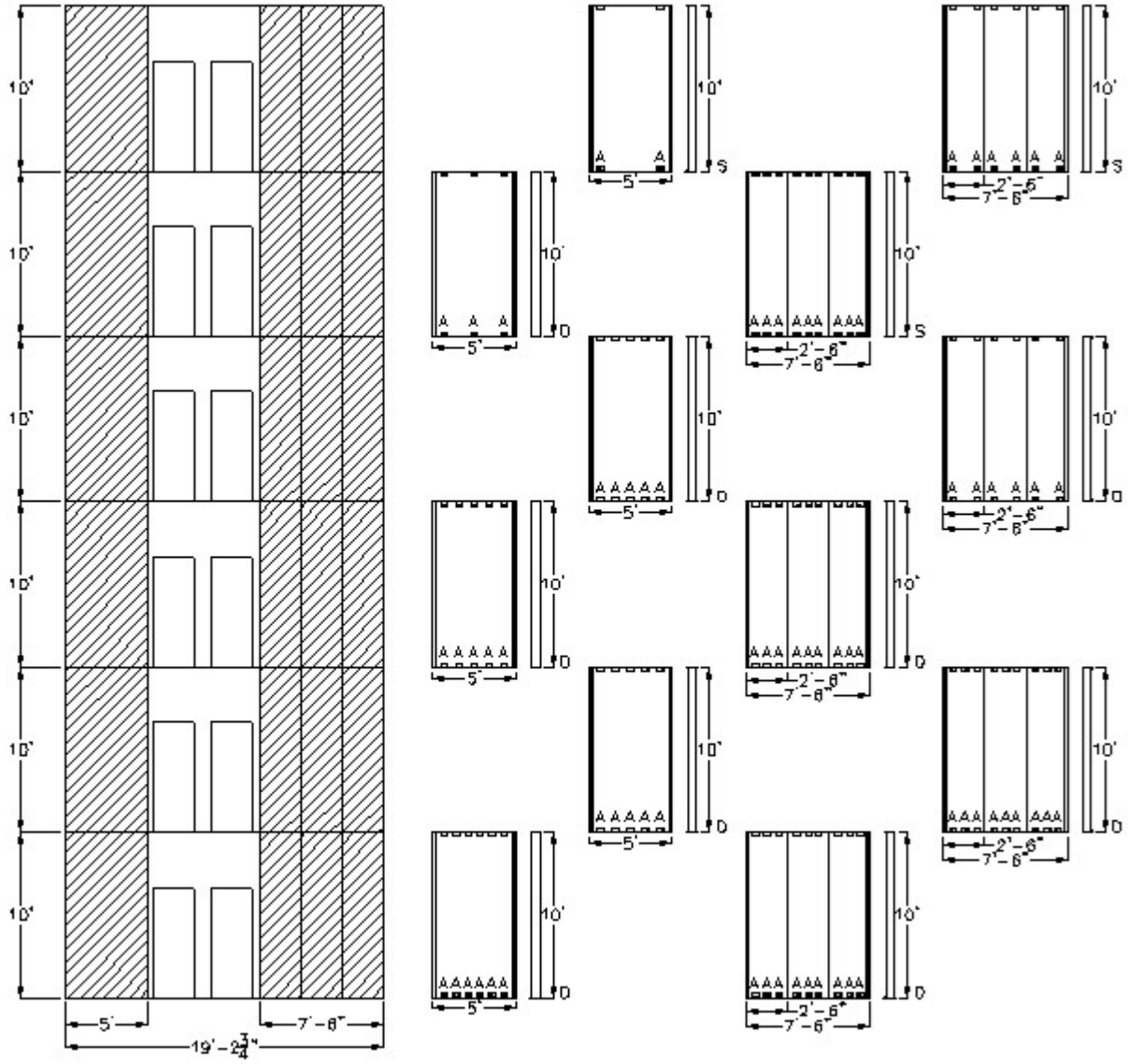


Figure H7. Example archetype design.

- There are no interpanel connectors for the single-panel case. For the multipanel configuration, the spacing of interpanel connectors is the same as for connectors at the base resulting in twice the number of connectors as used at the base.
- Overturning–applied unit shear:

  Applied unit shear is required for overturning calculations and is calculated based on the relative stiffness of each shear wall within the wall line, which is calculated based on Equation (5.1) provided in the design methodology.

  The applied unit shear values for the example wall stack are given in the following table:

	Applied unit shear (lb/ft)	
Story	(1) 5-ft panel	(3) 2.5-ft panel
6	436.7	562.3
5	1,523.3	1,124.1
4	2,505.5	1,486.4
3	2,535.9	2,217.7
2	2,552.9	2,697.0
1	2,882.4	2,716.5

- Design overturning force

  Design of CLT special shear walls and associated load path shall be in accordance with basic load combinations of ASCE 7-16 Section 2.3.6 (load combinations without overstrength) (ASCE 2016).

  For LRFD, the applicable load combinations are

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

$$(0.9 - 0.2S_{DS})D + \rho Q_E$$

  $S_{DS} = 1.0$

  $\rho = 0$

  $L = 0$

  $S = 0$

  D = only wall self-weight; 25.9 lb/ft^2 (interior wall)

  Looking at Figure H2, the floor panel orientation for gravity distribution is considered in the north–south direction. For the purpose of archetype design calculations in this project, the assumption is that the gravity is only considered in the direction of the orientation of the panel. Therefore, the only gravity load in this archetype example is self-weight of the walls because the walls are oriented parallel to the direction of the floor panels.

- Bearing capacity is based on the reinforced option mentioned in Appendix F of this document.

Overturning calculations for the 5-ft segment of the wall are shown in Table H8.

The design method requires the tie-down system to develop 1.15 times the nominal capacity of the shear wall. Overturning calculations were performed to determine the corresponding rod capacity. Table H9 gives the design values for overturning demand for each story. An example calculation for the first story is shown in Table H10. Similar calculations were performed for the remaining stories. Tensile forces were added at each level. The design overturning forces are summarized in Table H11.

Section 5.3.4 of the methodology requires the following:

- Rods at each level shall be designed for cumulative overturning tensile forces and bearing shall be provided at the floor level above each story.
- The nominal strength of the hold-down device shall not be less than required to resist the net uplift forces associated with development of 1.15 times the nominal unit shear capacity in accordance with Equation (5.2).

Tie-down design calculations are provided in Table H12, and bearing forces are calculated in Table H13. For the purpose of this single-panel example, the moment arm for the tie-down was assumed to be from the center of the compression zone to the tension edge of the panel. The actual moment arm would be in accordance with the details of the final design, i.e., center of tension force to center of compression zone.

Table H8—Overturning calculations for the 5-ft segment

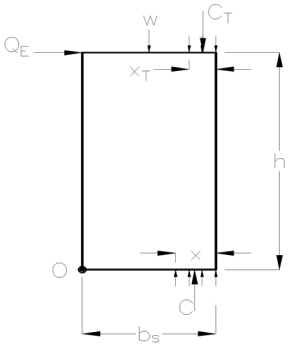
Story	Gravity load (kip/ft)	Unit shear (lb/ft)	End panel length (ft)	Thickness (in.)	Total compression zone (ft)	Applied OT moment (kip-ft)-shear only	Applied OT moment (kip-ft)-shear and gravity	Resisting moment (kip-ft)	Cumulative overturning demand (kip)	T force from applied OT moment (shear and gravity counteracting) for story (kip)	Cumulative T force from applied OT moment (shear and gravity counteracting) for story (kip)
6	0.26	436.7	5.0	4.13	0.11	21.84	26.37	26.37	5.33	3.97	3.97
5	0.26	1,523.3	5.0	4.13	0.47	76.16	107.08	107.08	22.47	15.55	19.52
4	0.32	2,505.5	5.0	6.88	0.36	125.27	238.01	238.01	49.40	25.46	44.97
3	0.39	2,535.9	5.0	9.63	0.35	126.80	371.59	371.59	77.02	25.63	70.60
2	0.45	2,552.9	5.0	12.38	0.37	127.65	507.15	507.14	105.37	25.76	96.36
1	0.45	2,882.4	5.0	12.38	0.49	144.12	659.16	659.17	138.67	29.57	125.93

Story	Gravity load (kip/ft)	1.15*v_n unit shear (lb/ft)	End panel length (ft)	T force from applied OT moment (shear and gravity counteracting) for story (kips)	Cumulative T force from applied OT moment (shear and gravity counteracting) for story (kips)
6	0.26	1,198.3	5.0	11.7	11.7
5	0.26	3,594.9	5.0	37.3	49.0
4	0.32	5,991.5	5.0	61.6	110.6
3	0.39	5,991.5	5.0	61.4	172.0
2	0.45	5,991.5	5.0	61.5	233.5
1	0.45	7,189.8	5.0	74.9	308.4

Table H9—Overturning demand: design values for each story

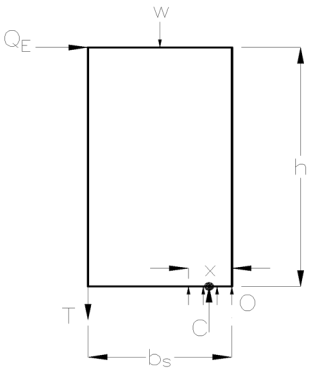
6th story shear wall
t = 4.125 in.
b = 5 ft
w = 25.93 lb/ft^2 × 5 ft ×10 ft = 1.3k; C_T = 0
Q_E = 436.7 plf × 5 ft = 2,183.6 lb
MO = v × b_s × h = 2,183.6 lb × 10 ft = 21.84k – ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 0.968$ kip/in^2 × $t \times x \times (b_s - x/2)$
x = 0.11 ft
C = 0.968 kip/in^2 × $t \times x$ × (12 in./ft) = 5.33k
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
T = 3.97k
$T_6 = T$ = 3.97k



5th story shear wall
t = 4.125 in.
b = 5 ft
w = 25.93 lb/ft^2 × 10 ft × 5 ft = 1.3k; C_T = 5.33k; x_T = 0.11 ft
Q_E = 1,523.3 plf × 5 ft = 7,616.5 lb
MO = v × b_s × h = 7,616.5 lb × 10 ft = 76.16k – ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 0.968$ kip/in^2 × $t \times x \times (b_s - x/2)$
x = 0.47 ft
C = 0.968 kip/in^2 × $t \times x$ × (12 in./ft) = 22.47k
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
T = 15.55k ⇒ T_5 = 15.55 + T_6 = 15.55 + 3.96 = 19.52k



4th story shear wall
t = 6.875 in.
b = 5 ft
w^a = 32.35 lb/ft^2 × 10 ft × 5 ft = 1.62k; C_T = 22.47k; x_T = 0.47 ft
Q_E = 2,505.5 plf × 5 ft = 12,527.3 lb
MO = v × b_s × h = 12,527.3 lb × 10 ft = 125.27k - ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 1.646$ kips/in^2 × $t \times x \times (b_s - x/2)$
x = 0.36 ft
C = 1.646 kips/in^2 × $t \times x$ × (12 in./ft) = 49.40k
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
T = 25.46k ⇒ T_4 = 25.46 + T_5 = 25.45 + 19.52 = 44.97k
[a]This value was adjusted from the one shown in Table A1 for 6.875-in. wall thickness

3rd story shear wall
t = 9.625 in.
b = 5 ft
w^a = 38.77 lb/ft^2 × 10 ft × 5 ft = 1.94k; C_T = 49.40k; x_T = 0.36 ft
Q_E = 2,535.9 plf × 5 ft = 12,679.6 lb
MO = v × b_s × h = 12,679.6 lb × 10 ft = 126.8k - ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 1.904$ kips/in^2 × $t \times x \times (b_s - x/2)$
x = 0.35 ft
C = 1.904 kips/in^2 × $t \times x$ × (12 in./ft) = 77.02k
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
T = 25.63k ⇒ T_3 = 25.63 + T_4 = 25.63 + 44.97 = 70.6k
[a]This value was adjusted from the one shown in Table A1 for 9.625-in. wall thickness

Table H9—Overturning demand: design values for each story—continued

2nd story shear wall
t = 12.375 in.
b = 5 ft
w^a = 45.18 lb/ft^2 × 10 ft × 5 ft = 2.26k; C_T = 77.02k; x_T = 0.35 ft
Q_E = 2,552.9 plf × 5 ft = 12,764.7 lb
$MO = v \times b_s \times h$ = 12,764.7 lb × 10 ft = 127.65k - ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 1.895$ kips/in$^2 \times t \times x \times (b_s - x/2)$
x = 0.37 ft
$C = 1.895$ kips/in$^2 \times t \times x \times$ (12 in./ft) = 105.37k
$vT \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
$T = 25.76k \Rightarrow T_2 = 25.76 + T_3 = 25.76 + 70.60 = 96.36k$
[a]This value was adjusted from the one shown in Table A1 for 12.375-in. wall thickness

1st story shear wall
t = 12.375 in.
b = 5 ft
w^a = 45.18 lb/ft^2 × 10 ft × 5 ft = 2.26k; C_T = 105.37k; x_T = 0.37 ft
Q_E = 2,882.4 plf × 5 ft = 14,412.0 lb
$MO = v \times b_s \times h$ = 14,412.0 lb × 10 ft = 144.12k - ft

The following equation is solved for x:
$v \times b_s \times h + 1.4 \times w \times b_s \times b_s/2 + C_T \times (b_s - x_T/2) = 1.895$ kips/in$^2 \times t \times x \times (b_s - x/2)$
x = 0.49 ft
$C = 1.895$ kips/in$^2 \times t \times x \times$ (12 in./ft) = 138.67k
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
$T = 29.57k \Rightarrow T_1 = 29.57 + T_2 = 29.57 + 96.36 = 125.93k$
[a]This value was adjusted from the one shown in Table A1 for 12.375-in. wall thickness

Table H10—Overturning demand: nominal values for each story

1st story shear wall
b = 5 ft
w = 48.15 lb/ft^2 × 10 ft × 5 ft = 2.26
$1.15V_N$ = 7,198.8 lb/ft × 5 ft = 35,949 lb
MO = $v \times b_s \times h$ = 35,949 lb × 10 ft = 359.49k – ft
x = 0.49 ft
$T \times (b_s - x/2) = v \times b_s \times h - 0.7 \times w \times b_s \times (b_s/2 - x/2)$
$T = 74.9k \Rightarrow T_1 = 74.9 + T_2 = 74.9 + 233.5 = 308.4k$

T_2 is the accumulated tensile load on the second story corresponding to the nominal capacities of the stories above the second story times 1.15.

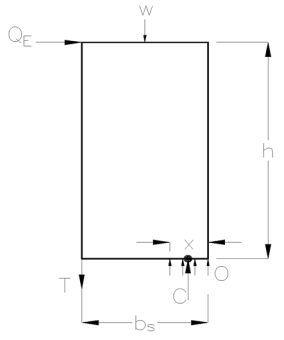


Table H11—Design overturning forces summary

			Design		Nominal	
Story	Cumulative overturning demand (kip)	Total compression zone (ft)	T force from applied OT moment (shear and gravity counteracting) for story (kip)	Cumulative T force from applied OT moment (shear and gravity counteracting) for story (kip)	T force from applied OT moment (shear and gravity counteracting) for story (kip)	Cumulative T force from applied OT moment (shear and gravity counteracting) for story (kip)
6	5.33	0.11	3.97	3.97	11.7	11.7
5	22.47	0.47	15.55	19.52	37.3	49.0
4	49.40	0.36	25.46	44.97	61.6	110.6
3	77.02	0.35	25.63	70.60	61.4	172.0
2	105.37	0.37	25.76	96.36	61.5	233.5
1	138.67	0.49	29.57	125.93	74.9	308.4

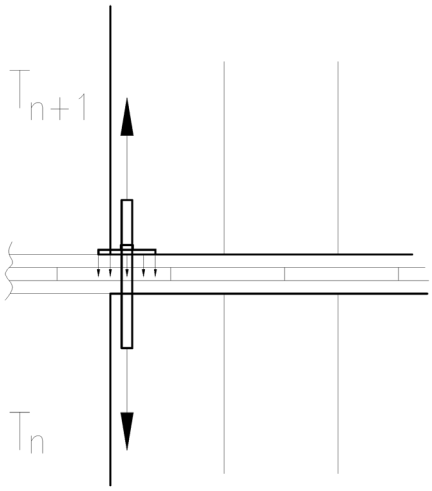
Table H12—Tie-down rod design

							Design		Ultimate capacity design		Drift design	
Story	No. of rods	Diam. (in.)	Nominal area (in^2)	Net area (in^2)	F_y (lb/in^2)	F_u (lb/in^2)	Design strength, $ØR_n$ (kip)	T (kip) demand	Rod ultimate strength, R_n (kip)	T corresponding to wall ultimate capacity	T (kip)[a]	Rod elongation (in.)
6	1	½	0.196	0.142	92,000	120,000	13.25	4.0	17.7	11.67	3.8	0.11
5	4	½	0.196	0.142	92,000	120,000	53.01	19.5	70.7	48.96	19.1	0.14
4	4	3/4	0.442	0.334	92,000	120,000	119.28	45.0	159.0	110.59	44.4	0.14
3	4	7/8	0.601	0.462	92,000	120,000	162.36	70.6	216.5	172.02	69.7	0.16
2	4	1	0.785	0.606	92,000	120,000	212.06	96.4	282.7	233.51	95.1	0.16
1	4	1 1/8	0.994	0.763	92,000	120,000	268.39	125.9	357.8	308.38	124.4	0.17

[a]This was calculated based on 1.0D instead of 0.7D used for the design T calculation.

Table H13—Bearing forces at each story

No. of Stories	No. of rods	T corresponding to wall ultimate capacity LRFD cumulative tension (kip)	Bearing/plate (kip)
6	1	11.7	11.7
5	4	49.0	37.3
4	4	110.6	61.6
3	4	172.0	61.4
2	4	233.5	61.5
1	4	308.4	74.9



The following is the deflection calculation for the first story:

$$\Delta = \frac{576 v b_s h^3}{\mathrm{EI}_{\mathrm{eff}}} + 3\Delta_{\text{nail slip,h}} + 2\Delta_{\text{nail slip,v}} \frac{h}{b_s} + \Delta_a \frac{h}{b}$$

Note that shear wall deflection calculations do not include the panel shear deformation component of deflection – the panel shear deformation is typically less than 5% of the total deflection and, for purposes of this example, is not considered to make a practical difference in the final design.

Bending:

$$\delta_b = 576 v b_s h^3/(\mathrm{EI}_{\mathrm{eff}})$$

$$\mathrm{EI}_{\mathrm{eff}} = \left[1 - \left(1 - \frac{E_{90,T}}{E_{0,L}}\right) \frac{a_{m-2} - a_{m-4} + \cdots \pm a_1}{a_m}\right] E_{0,L} \frac{b_{panel}^3 \times a_m}{12}$$

$$\mathrm{EI}_{\mathrm{eff}} = \left[1 - \left(1 - \frac{40{,}000 \text{ lb/in}^2}{1{,}400{,}000 \text{ lb/in}^2}\right) \frac{9.625 \text{ in.} - 6.875 \text{ in.} + 4.125 \text{ in.} - 1.375}{12.375}\right] 1{,}400{,}000 \text{ lb/in}^2 \times \frac{(5 \text{ ft} \times 12 \text{ in/ft})^3 \times 12.375 \text{ in.}}{12} = 1.772E11 \text{ lb-in}^2$$

$$\delta_b = 576 \times 2{,}882.4 \text{ lb/ft} \times 5 \text{ ft} \times \frac{(10 \text{ ft})^3}{1.772E11 \text{ lb-in}^2} = 0.05 \text{ in.}$$

Shear (sliding):

$$\Delta_{\text{nail slip, h}} = \frac{v_n}{6{,}700} = \frac{150.13}{6{,}700} = 0.022 \text{ in.}$$

$$V_n = \frac{2{,}882.4 \text{ lb/ft} \times 5 \text{ ft}}{2 \times 6 \text{ conn} \times 8 \text{ nails/conn}} = 150.13 \text{ lb/nail}$$

$$\delta_s = 3\Delta_{\text{nail slip, h}} = 3 \times 0.022 \text{ in.} = 0.067 \text{ in.}$$

Panel rotation caused by vertical joint slip:

$$\Delta_{\text{nail slip, v}} = 0$$

Rigid body overturning (rocking):

Δ_1 = elastic deformation in compression toe

Δ_2 = elastic deformation in tension side

Δ_3 = rod elongation

E_0 = modulus of elasticity = 1,400,000 lb/in^2

$E_{perp} = E_0/30 = 1{,}400{,}000 \text{ lb/in}^2/30 = 46{,}666.67 \text{ lb/in}^2$

$$\Delta_1 = \frac{PL}{AE} = \frac{C \times t_{\text{floor}}}{A_C \times E_{\text{perp}}}$$

$$\Delta_2 = \frac{PL}{AE} = \frac{C \times t_{\text{floor}}}{A_C \times E_{\text{perp}}}$$

However, because of reinforcement, Δ_1 and Δ_2 are both zero.

$$\Delta_3 = \frac{PL}{A_{\text{net}}E} = \frac{T \times L}{A_{\text{rod}} \times E} = \frac{124.39k \times 120 \text{ in.}}{4 \times 0.763 \text{ in}^2 \times 29{,}000 \text{ ksi}} = 0.168 \text{ in.}$$

$$\Delta_a = (\Delta_1 + \Delta_2 + \Delta_3)\frac{\sum b_s}{\bar{b}} = (0.0 + 0.0 + 0.17) \times \frac{5}{(5 - 0.49/2)} = 0.177 \text{ in.}$$

$$\delta_r = \Delta_a \frac{h}{b} = 0.177 \times 2 = 0.35$$

$$\delta_{sw} = \delta_b + \delta_s + \delta_R = 0.05 + 0.07 + 0.35 = 0.47 \text{ in.}$$

$$C_d \delta_{sw} = 3 \times 0.47 \frac{\text{in.}}{120 \text{ in.}} = 1.17\%$$

It is important to note that this archetype example, similar to any design problem, was performed in an iterative manner in which variables such as number of connectors, wall thicknesses, tie-down numbers, and sizes were refined. As part of this iterative procedure, deformation compatibility (equal shear wall deflection) was checked for shear walls in the wall line. This was used to distribute shear to the shear wall segments and led to the final design story shear capacity.

CLT walls not designated as part of the seismic force resisting system are required to meet requirements of Section 5.2-3 of the design methodology to provide load displacement response similar to the designated CLT shear walls.

Deflection calculation values are given in Tables H14 through H17 for rocking, sliding, bending, and summary, respectively.

Archetype design properties are summarized in Table H18. Based on the final design, the following CUREE

Table H14—Deflection calculation: rocking component

Modulus of elasticity, E_o (lb/in^2)	E_perp, E/30 (lb/in^2)	Modulus of elasticity, E90 (lb/in^2)	E_perp, E90/30 (lb/in^2)	Floor thickness (in.)	Elastic comp. toe fc_perp deformatin (in.)	Elastic tension side fc_perp deformatin (in.)	Rod elongation (Table 23) (in.)[a]	h/b	Δ_a, Total vertical elongation of the wall anchorage system[a]	$\Delta_a h/b$
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.11	2.0	0.11	0.22
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.14	2.0	0.14	0.29
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.14	2.0	0.14	0.29
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.16	2.0	0.16	0.32
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.16	2.0	0.16	0.34
1,400,000	46,666.67	1,200,000	40,000	6.65	0.00	0.00	0.17	2.0	0.17	0.35

[a]Rod elongation scaled to reflect anchorage deformation at the panel edge.

Table H15—Deflection calculation: sliding component

No. of nails/ft	Load per nail, v_n (lb)	Nail diameter, D (in.)	Sliding deflection $v_n/(135{,}000\ D^{1.5})$ (in.)	CLT panel sliding caused by horizontal joint slip (in.)	CLT panel aspect ratio	CLT panel rotation caused by vertical joint slip (in.)	Sliding + rotation caused by joint slip (in.)
3.2	136.48	0.135	0.02	0.06	2	0.00	0.06
9.6	158.68	0.135	0.02	0.07	2	0.00	0.07
16	156.59	0.135	0.02	0.07	2	0.00	0.07
16	158.50	0.135	0.02	0.07	2	0.00	0.07
16	159.56	0.135	0.02	0.07	2	0.00	0.07
19.2	150.13	0.135	0.02	0.07	2	0.00	0.07

Table H16—Deflection calculation: bending component

Number of layers	I, $b^3a_m/12$ (in^4)	$(EI)_{eff}$ (lb-in^2)	Deflection caused by bending $Ph^3/3EI$ (in.)
3	74,250	7.029E+10	0.02
3	74,250	7.029E+10	0.06
5	123,750	1.059E+11	0.07
7	173,250	1.416E+11	0.05
9	222,750	1.772E+11	0.04
9	222,750	1.772E+11	0.05

Table H17—Deflection calculation summary

Deflection caused by bending $Ph^3/3EI$ (in.)	Sliding + rotation caused by joint slip (in.)	$\Delta_a h/b$	Total deflection (in.)	C_d = 3, $C_d \times \delta$ (in.)	Drift (%)[a]
0.02	0.06	0.22	0.30	0.91	0.75
0.06	0.07	0.29	0.43	1.28	1.07
0.07	0.07	0.29	0.42	1.27	1.06
0.05	0.07	0.32	0.45	1.34	1.12
0.04	0.07	0.34	0.45	1.35	1.13
0.05	0.07	0.35	0.47	1.41	1.17

[a]The interstory drift ratio is checked against the drift limit of ASCE 7, which is 2%.

Table H18—Archetype example design properties

		Key archetype design parameters							
		Panel aspect ratio	Seismic design criteria						
No.	Archetype ID		Gravity	SDC	T (s)	T_1 (s)	V_b (kip)	W (kip)	S_{MT} (g)
52	4_3_6_1_MR_HG_DX_LP	Mixed	High	D_{max}	0.604	0.72	34.79	105	1.49

parameters, shown in Table H19, were used for static pushover and dynamic analysis.

Monotonic static pushover analysis was performed for the archetype using the inverted triangular load distribution associated with the EFL analysis. Loads were applied proportionate to the fundamental mode of the structure. The pushover curve for the example archetype is shown in Figure H8 with the red line representing $0.8V_{max}$. From inspection of Figure H8, one can see that V_{max} is 104.99 kip and δ_u, the displacement corresponding to 80% post peak capacity, is 23.52 in. With the calculated base shear, V_b, of 34.79 kip, Ω is calculated to be 3.02 for this archetype model.

As explained in Section 6.2 of this document, incremental dynamic analysis (IDA) was performed to determine the median collapse intensity, SCT. An IDA plot and collapse fragility for the example archetype are shown in Figure H9a and H9b, respectively. The latter was constructed based on 4.5% interstory drift limit discussed in Section 7.1. Looking at the Figure H9, SCT = 3.5 g, where the collapse margin ratio (CMR) taken as the ratio of SCT and MCE spectral acceleration, SMT, is calculated as 3.5 g/1.49 g = 2.35. A summary of the static and dynamic analyses are provided in Table H20. The CMR ratio was then adjusted based on the SDC and period-based ductility, μ_T, obtained from a pushover analysis. The determination of an acceptable adjusted CMR (ACMR) requires total collapse uncertainty, which was calculated based on the explanation provided in Section 7.2 of this document. The values for β_{DR}, β_{TD}, β_{MDL}, and β_{TD} were 0.2, 0.2, 0.2, and 0.4, respectively, which resulted in β_{TOT} of 0.529. This corresponds to an ACMR20% of 1.56.

Table H19—CUREE parameters used for example archetype

Configuration														
Panel length (ft)	No. of panels	Connector type	No. of connectors per panel	S/D	K_0 (lb/in.)	F_0 (lb)	F_1 (lb)	r_1	r_2	r_3	r_4	Δ_u (in.)	α	β
5	1	A3	6	D	24,000	51,504	6,600	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	5	D	20,000	42,920	5,500	0.05	–0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	3	D	12,000	33,400	3,000	0.1	–0.4	1.05	0.05	4	0.6	1.02
5	1	A3	2	S	5,000	8,760	1,500	0.05	–0.15	0.8	0.05	3.25	0.75	1.02
2.5	3	A3	2	D	26,500	36,500	5,100	0.075	–0.125	0.825	0.05	6.12	0.75	1.05
2.5	3	A3	2	S	13,250	18,250	2,550	0.075	–0.125	0.825	0.05	6.12	0.75	1.05
2.5	3	A3	3	D	39,750	54,750	7,650	0.075	–0.125	0.825	0.05	6.12	0.75	1.05
2.5	3	A3	3	S	19,875	27,375	3,825	0.075	–0.125	0.825	0.05	6.12	0.75	1.05

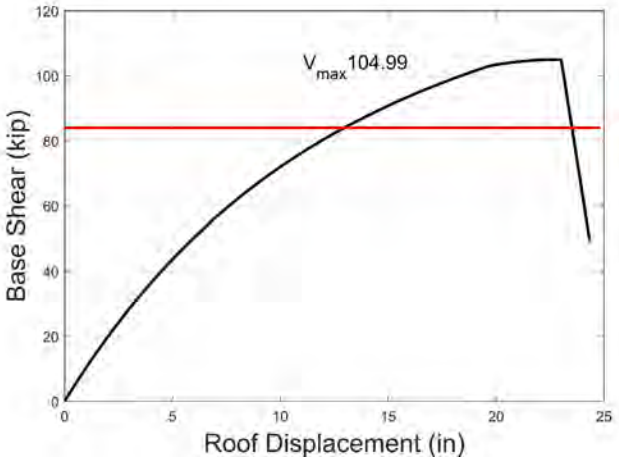
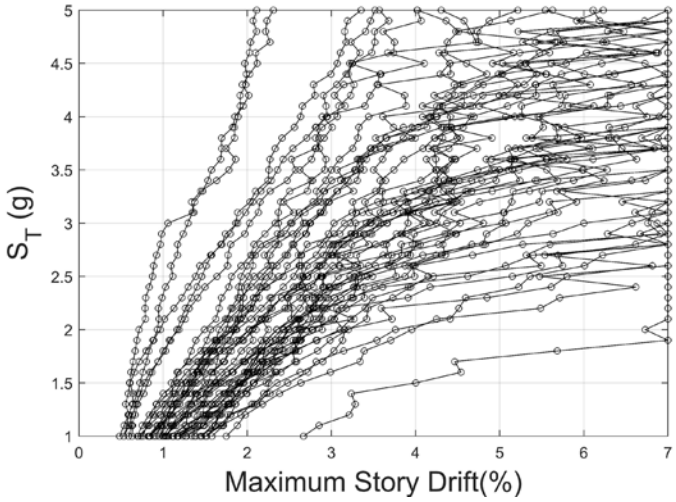
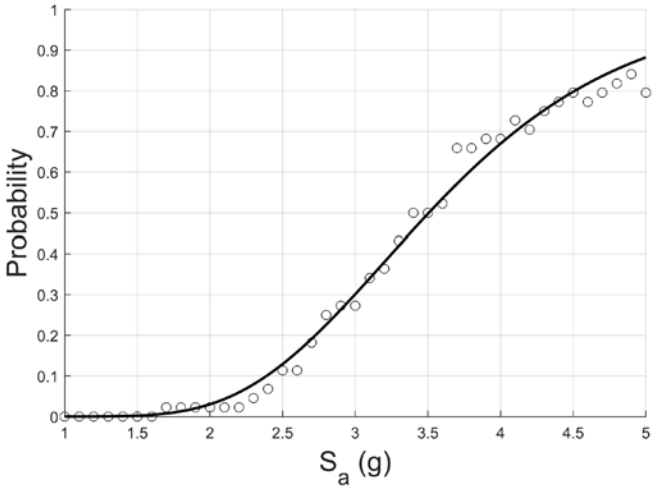


Figure H8. Archetype 52, R = 3, static pushover.



(a)



(b)

Figure H9. Archetype 52, R = 3, 2% damping, (a) IDA plot (b) collapse fragility based on 4.5% interstory drift limit.

Table H20—Summary of collapse results for archetype example

		Design configuration			Collapse margin parameters						Acceptance check	
No.	Archetype ID	Panel aspect ratio	Gravity	Seismic SDC	Ω	μ_T	S_{CT} (g)	CMR	SSF	ACMR	Acceptable ACMR	Pass/fail
52	4_3_6_1_MR_HG_DX_LP	Mixed	High	D_{max}	3.02	3.42	3.51	2.35	1.22	2.86	1.56	Pass

Appendix I: Archetype Results

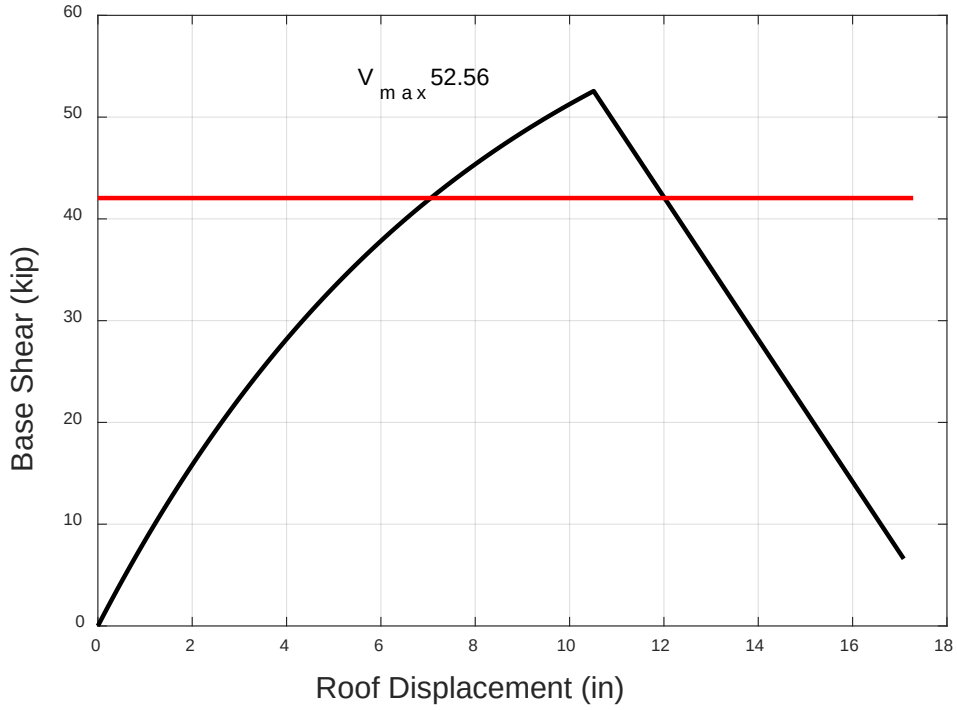


Figure I1. Archetype 01, R = 3, static pushover.

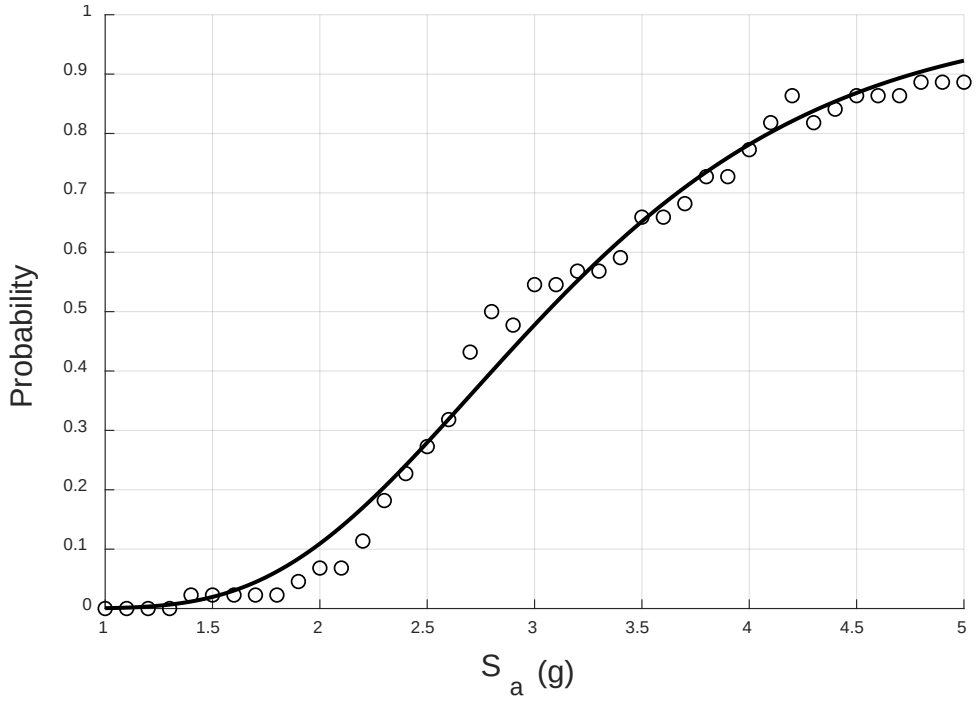


Figure I2. Archetype 01 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

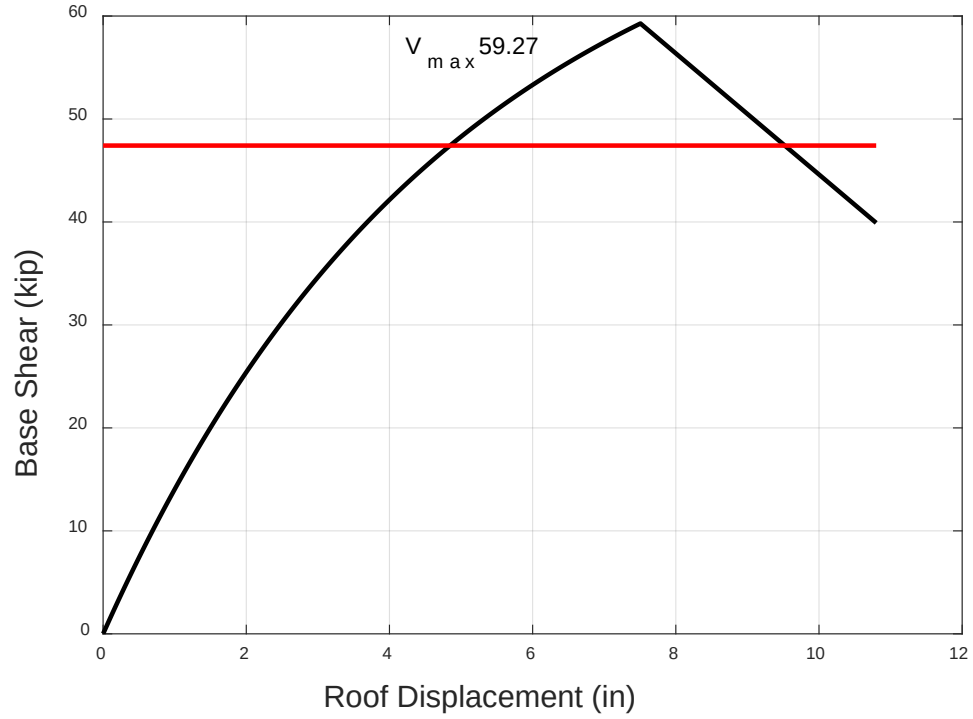


Figure I3. Archetype 02, R = 3, static pushover.

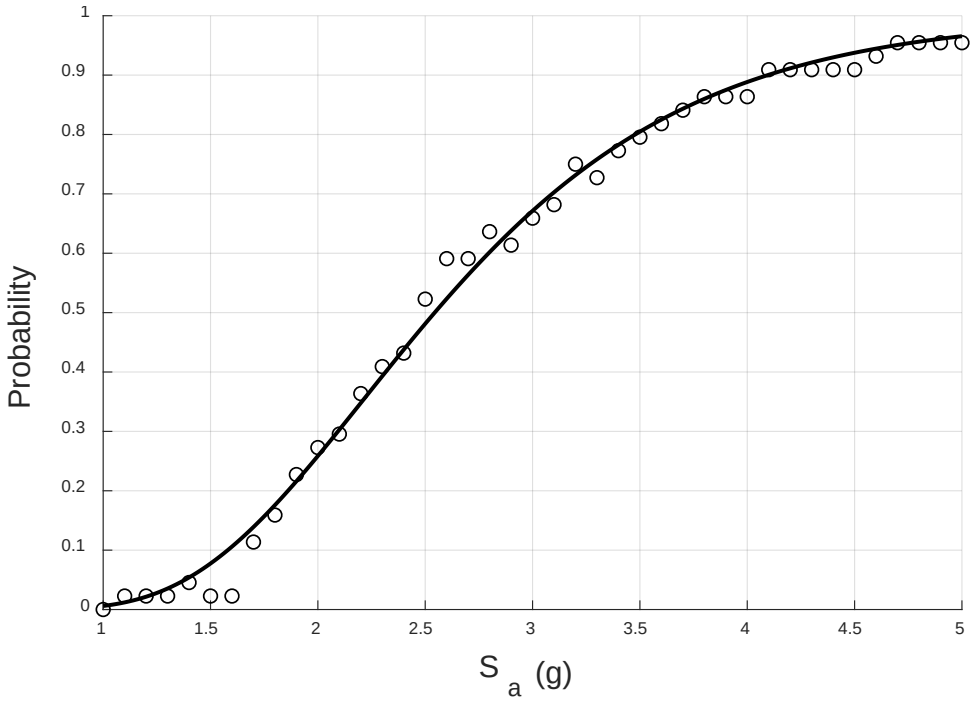


Figure I4. Archetype 02 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

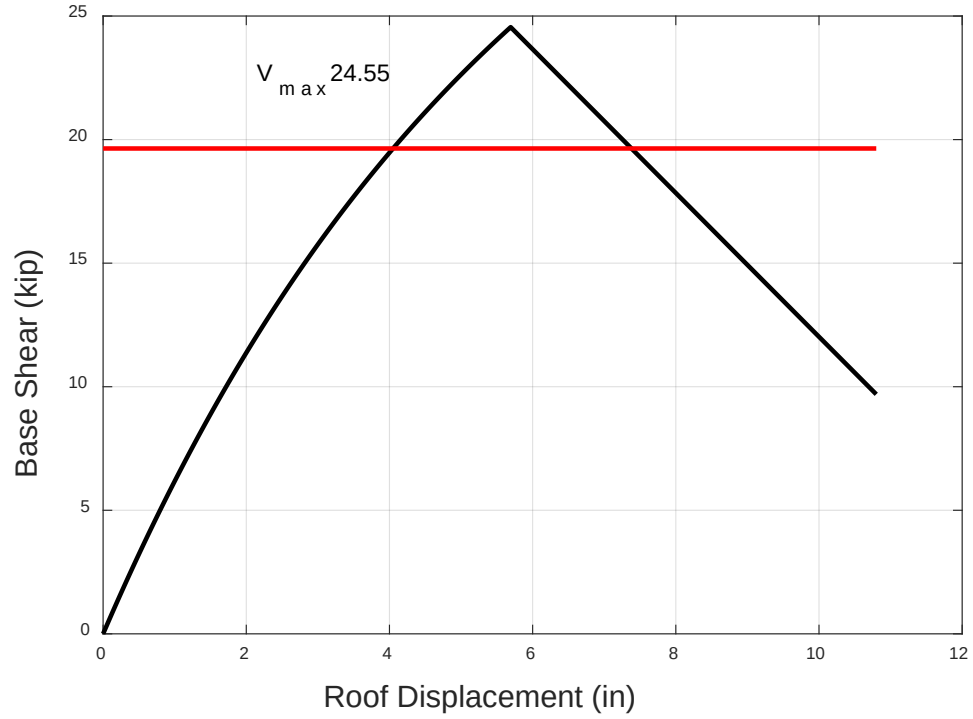


Figure I5. Archetype 03, R = 3, static pushover.

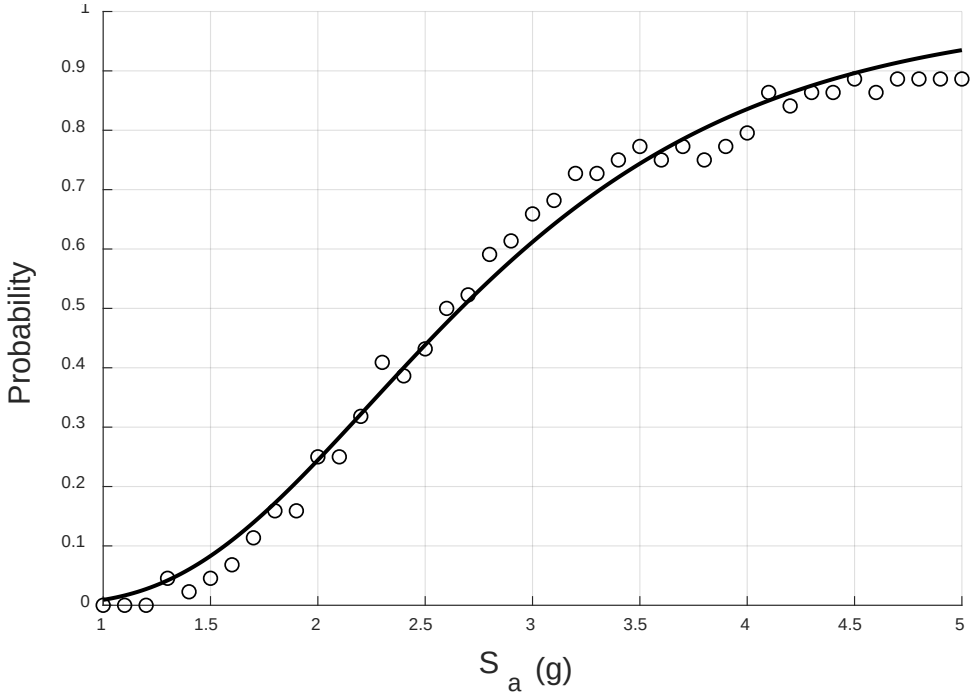


Figure I6. Archetype 03 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

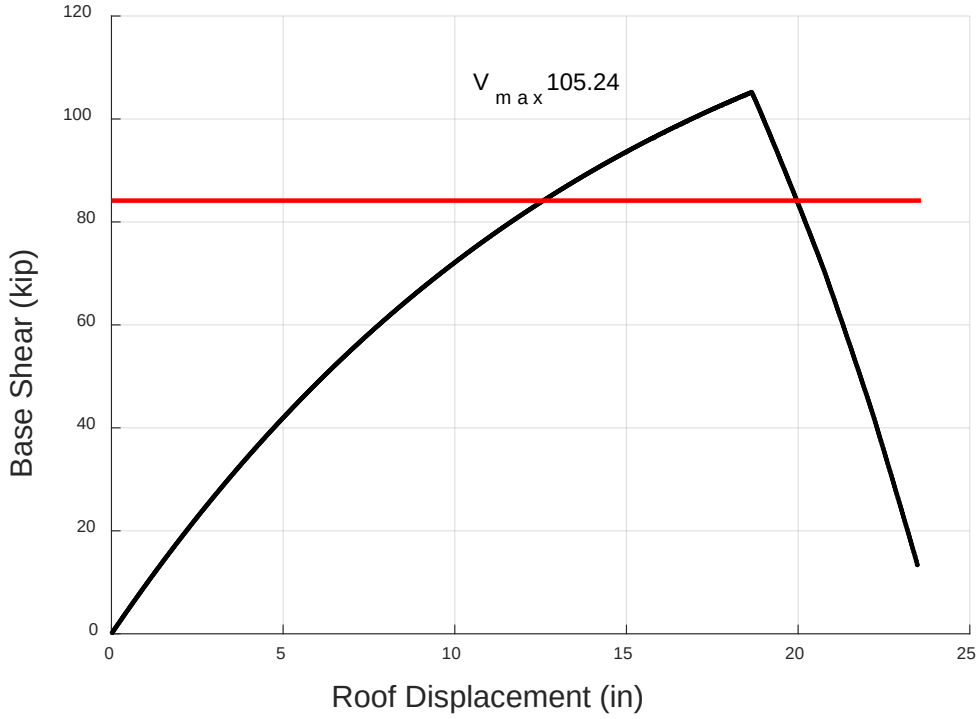


Figure I7. Archetype 04, R = 3, static pushover.

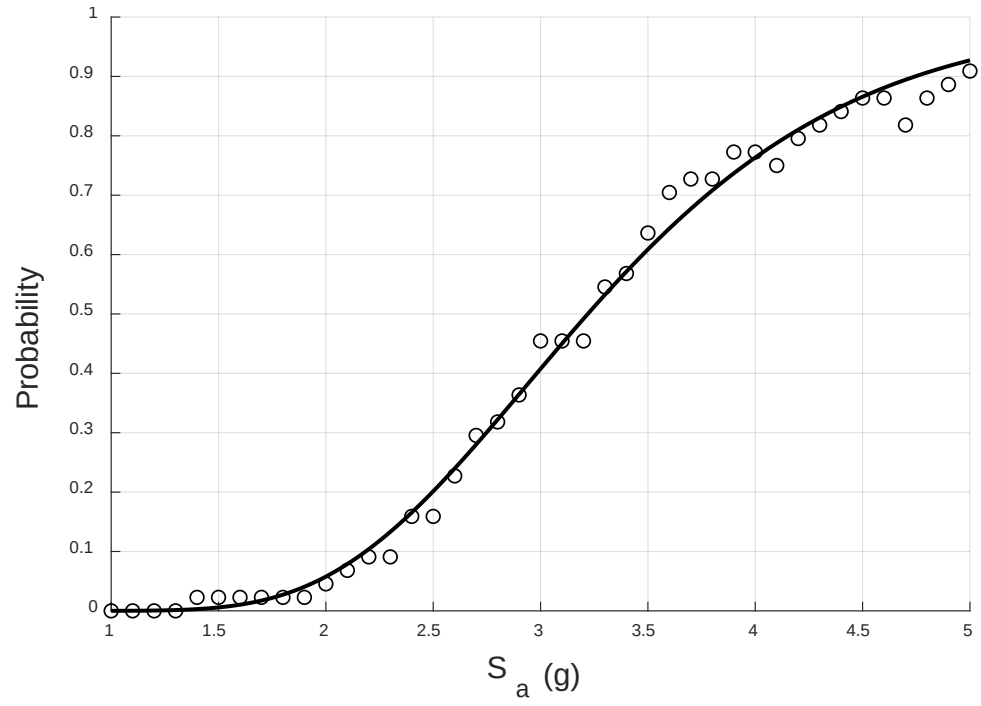


Figure I8. Archetype 04 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

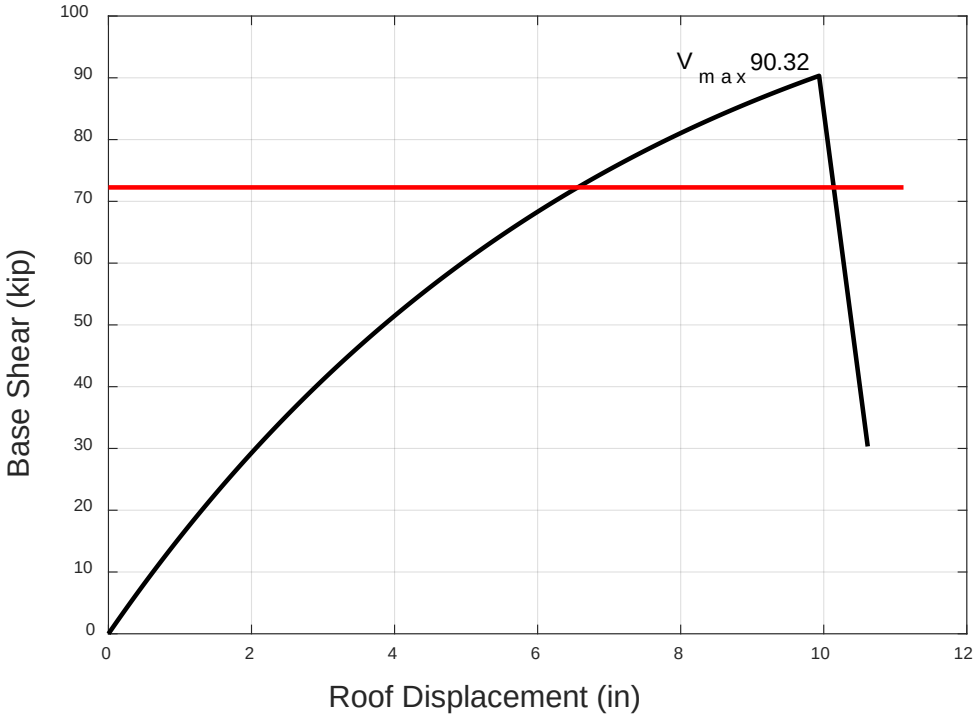


Figure I9. Archetype 05, R = 3, static pushover.

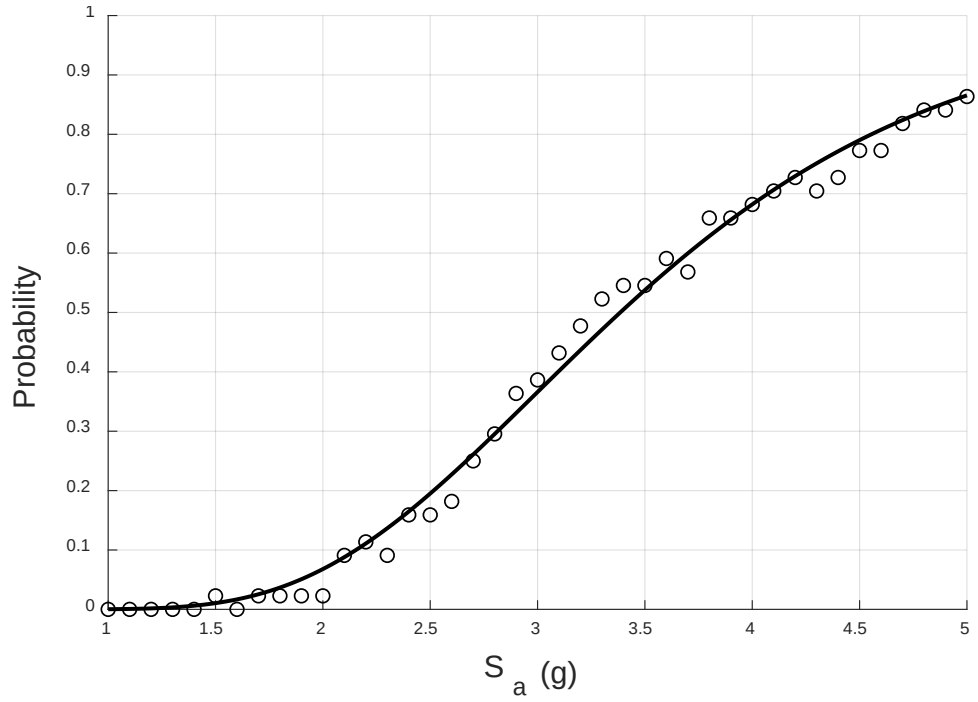


Figure I10. Archetype 05 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

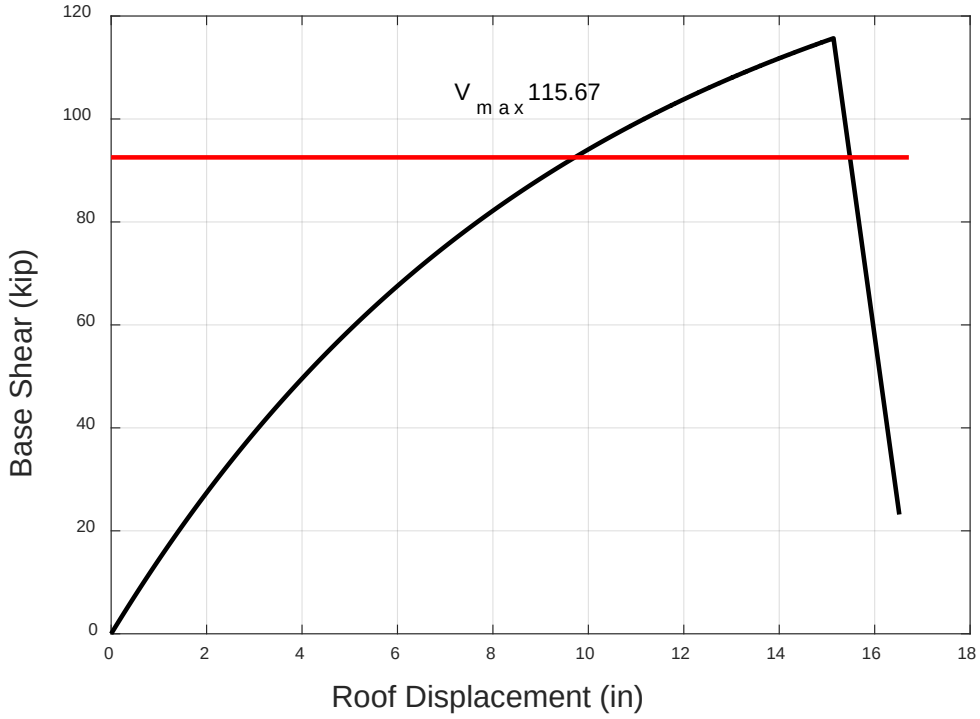


Figure I11. Archetype 06, R = 3, static pushover.

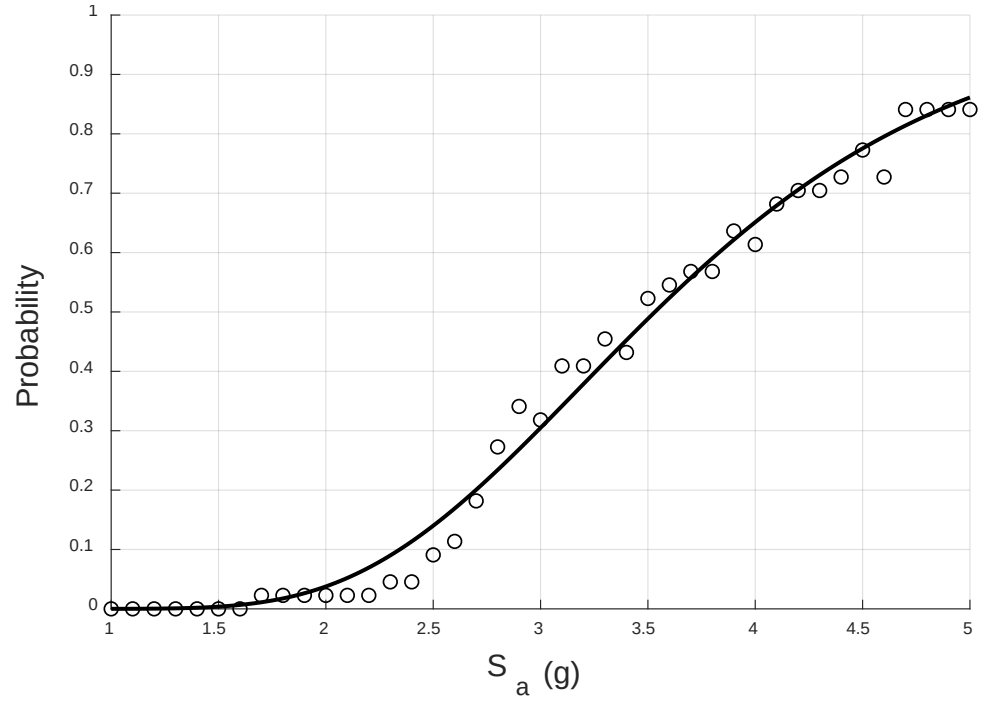


Figure I12. Archetype 06 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

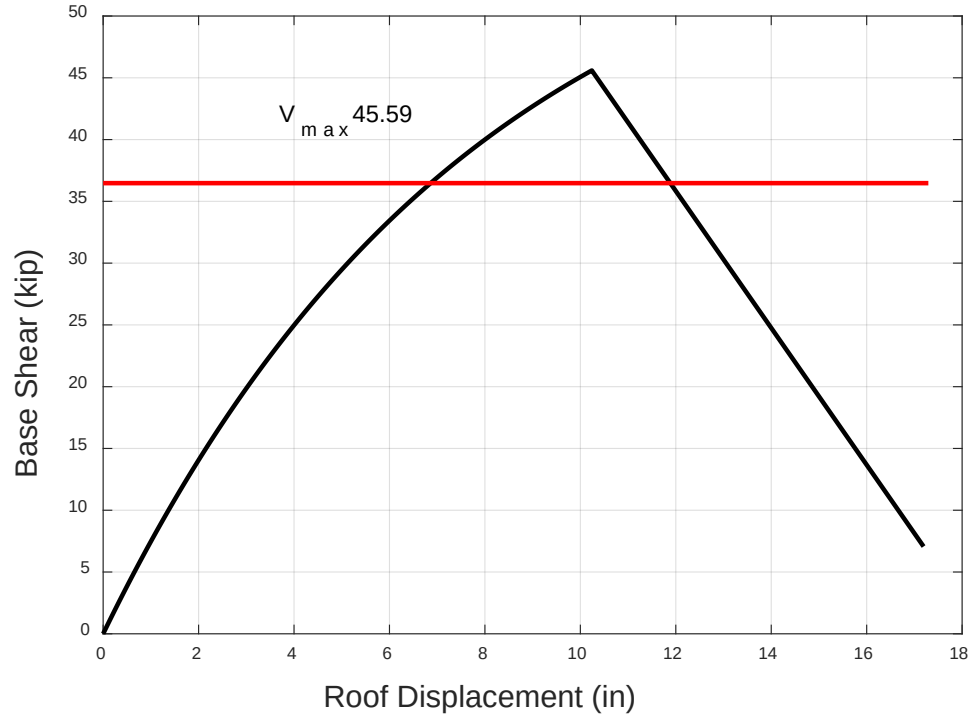


Figure I13. Archetype 13, R = 3, static pushover.

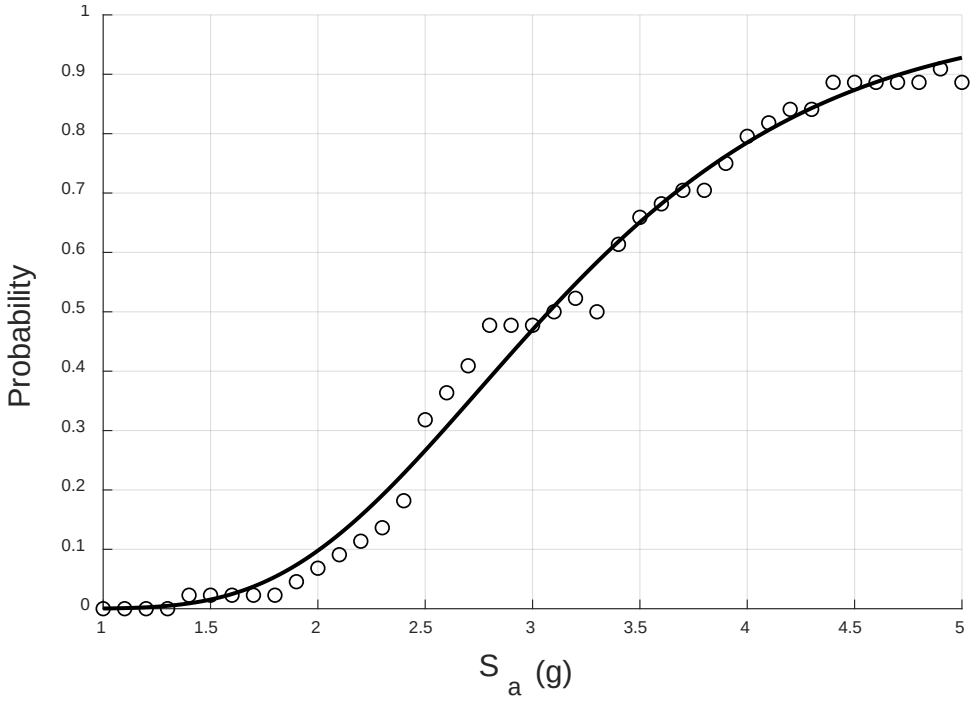


Figure I14. Archetype 13 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

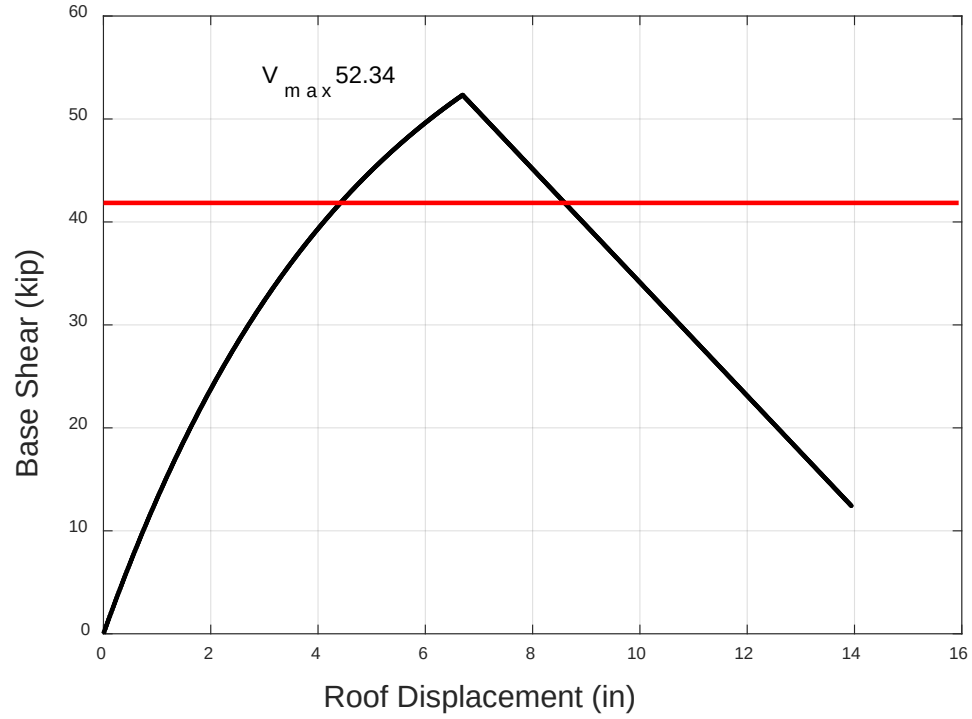


Figure I15. Archetype 14, R = 3, static pushover.

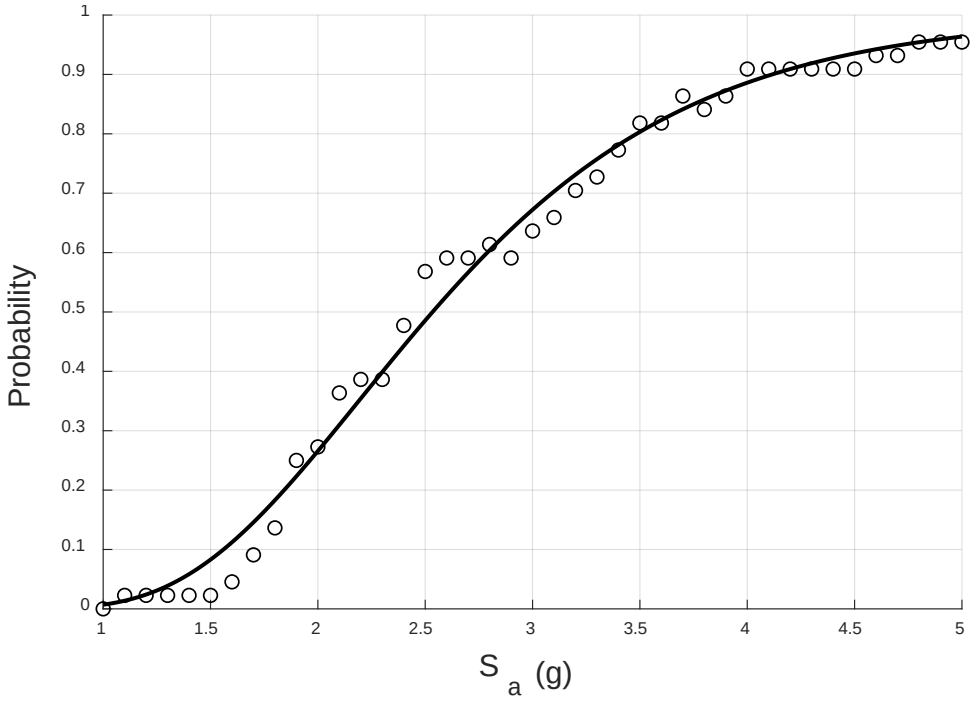


Figure I16. Archetype 14 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

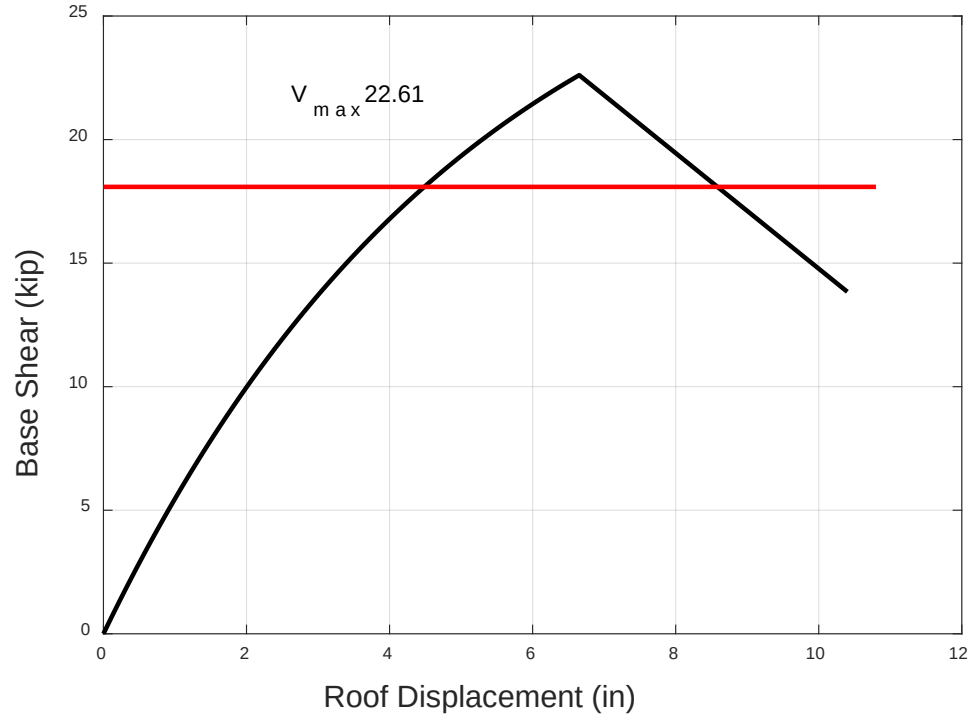


Figure I17. Archetype 15, R = 3, static pushover.

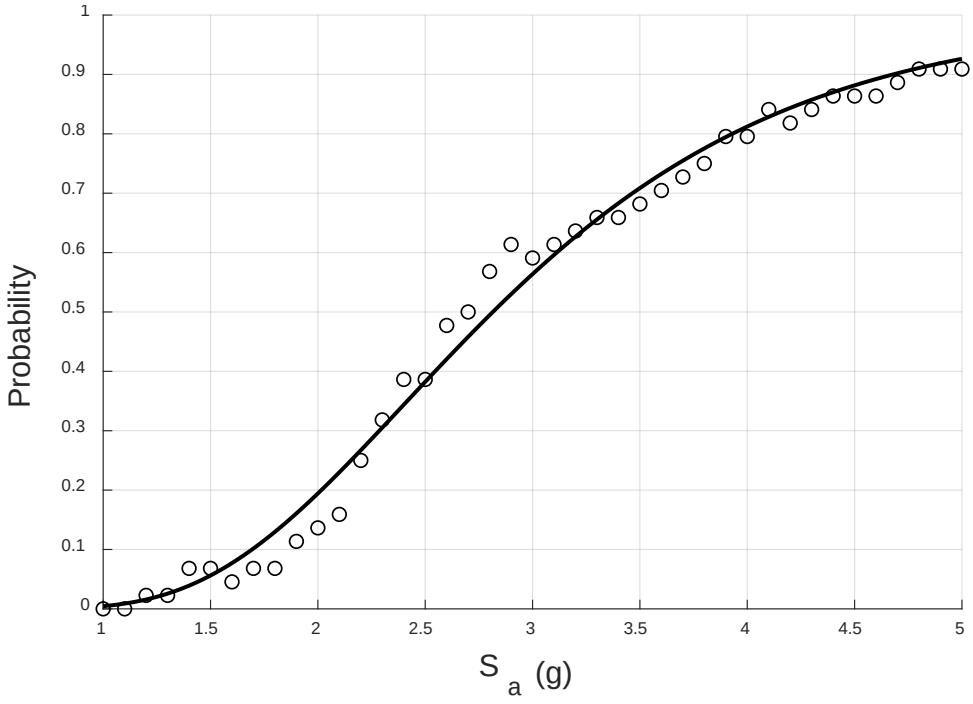


Figure I18. Archetype 15 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

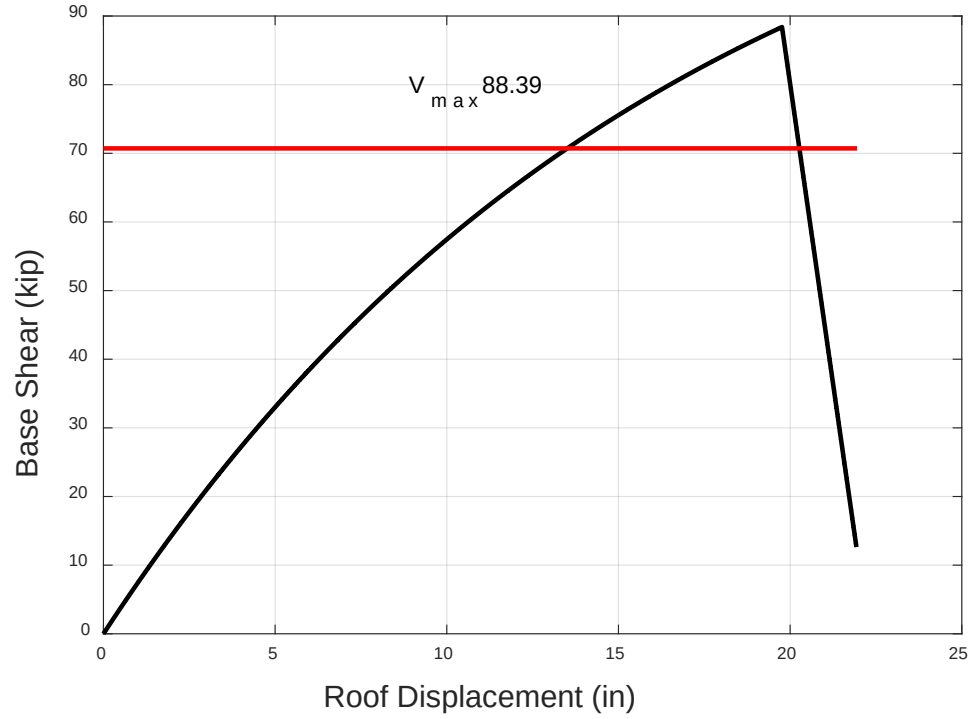


Figure I19. Archetype 16, R = 3, static pushover.

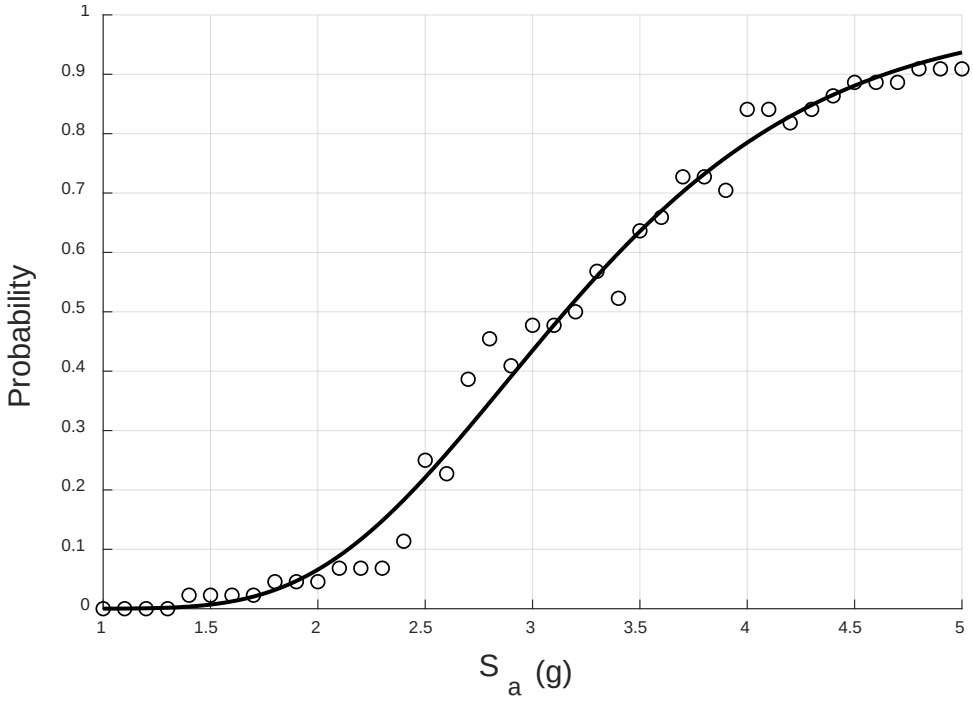


Figure I20. Archetype 16 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

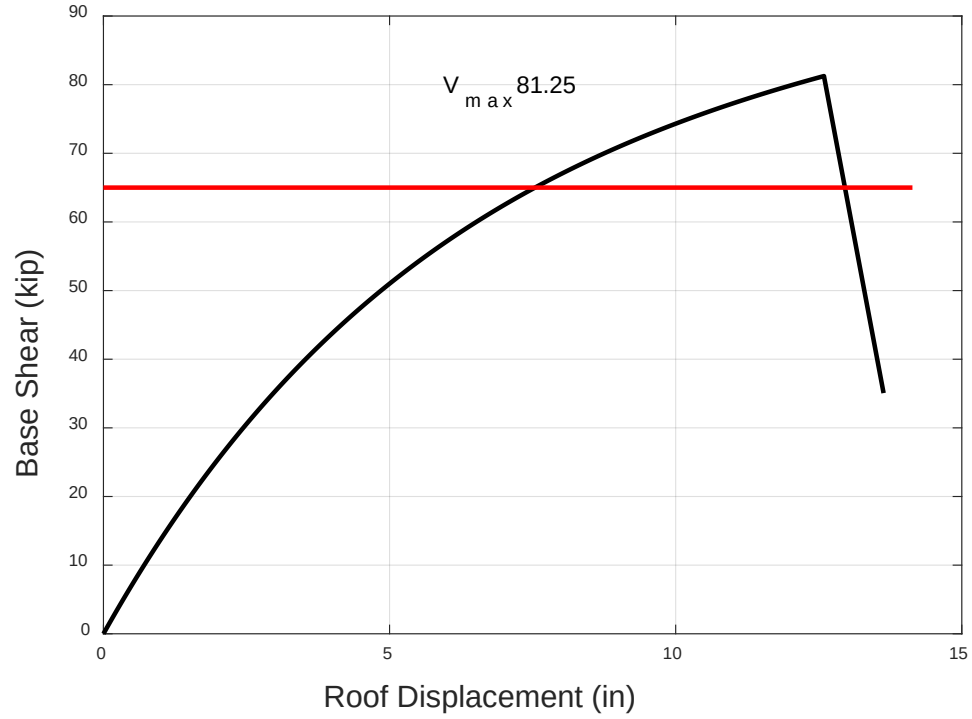


Figure I21. Archetype 17, R = 3, static pushover.

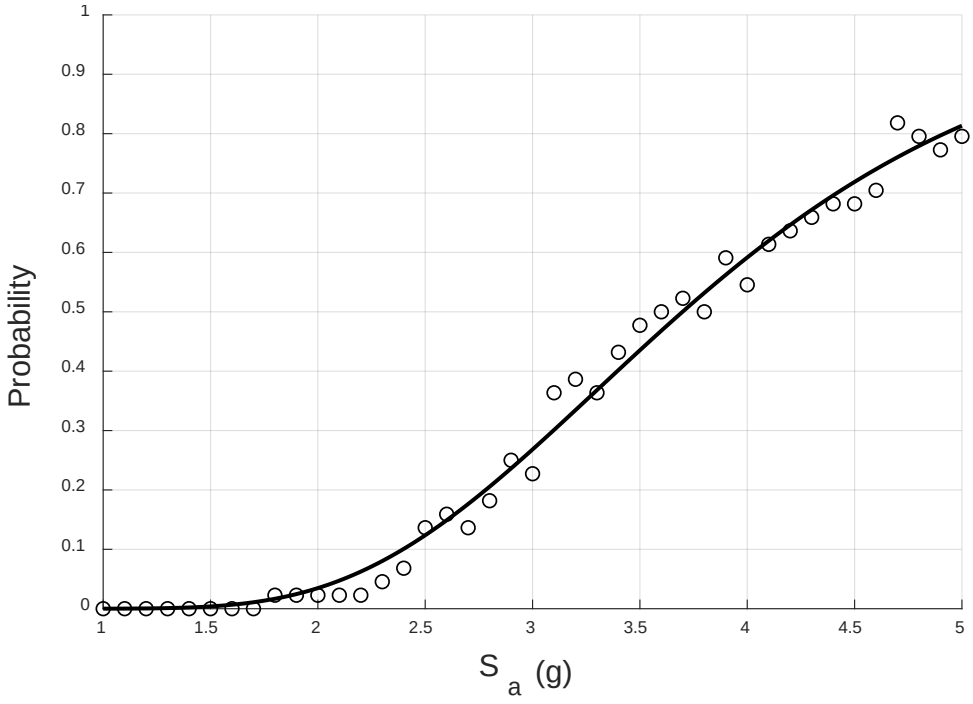


Figure I22. Archetype 17 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

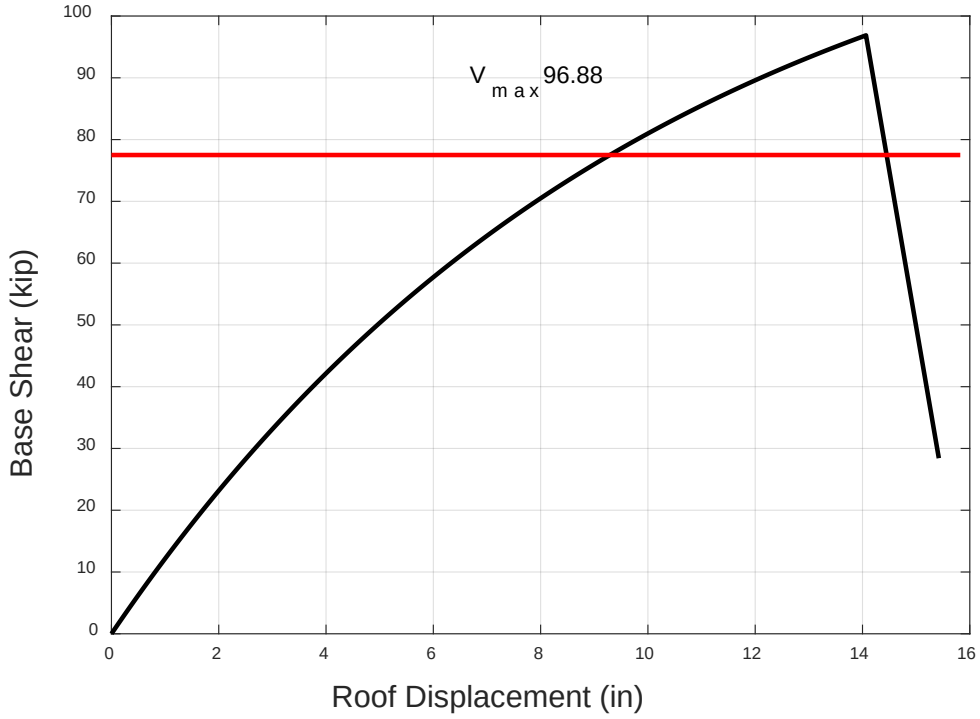


Figure I23. Archetype 18, R = 3, static pushover.

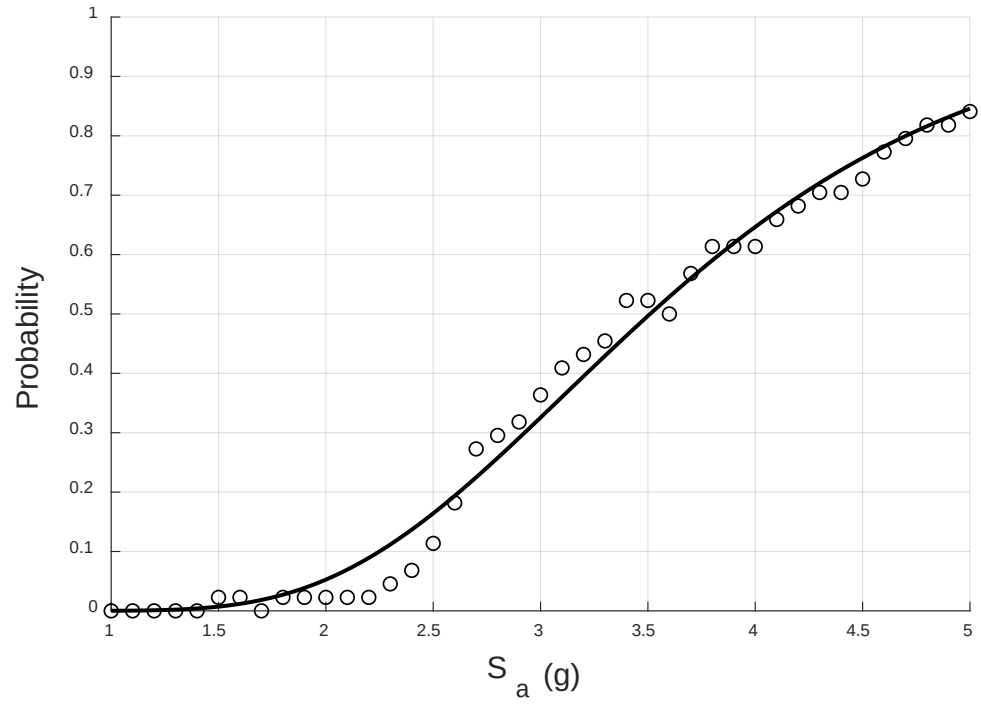


Figure I24. Archetype 18 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

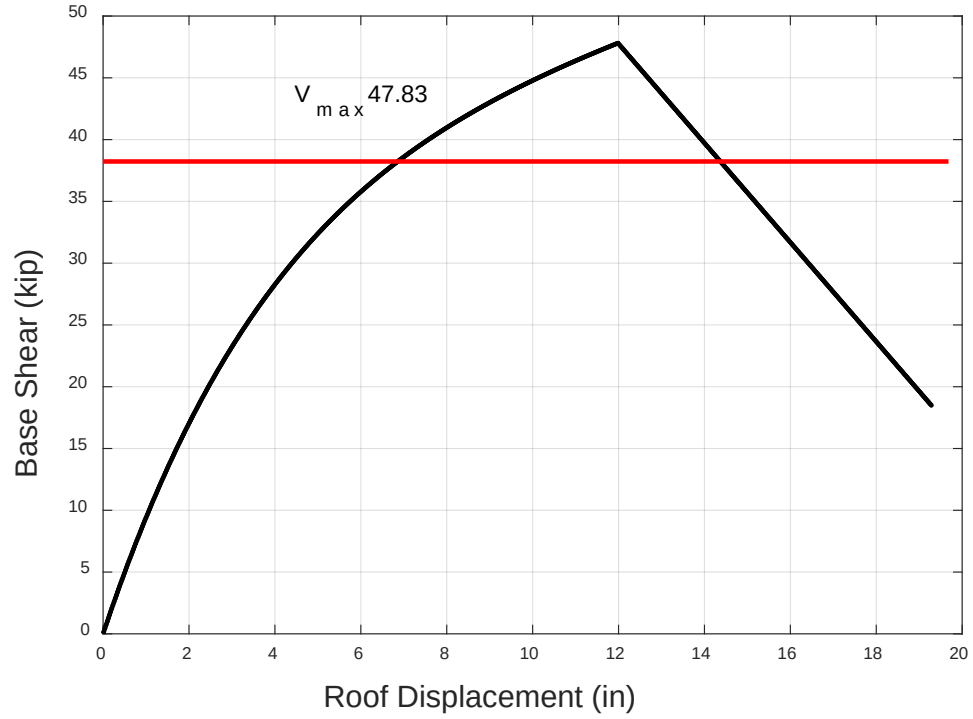


Figure I25. Archetype 25, R = 3, static pushover.

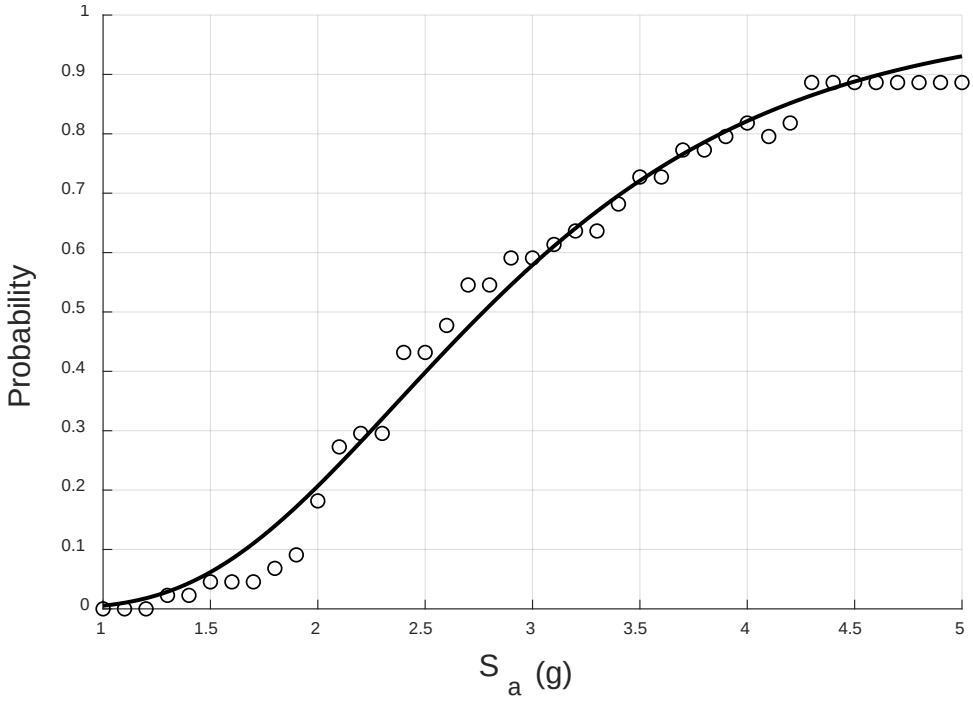


Figure I26. Archetype 25 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

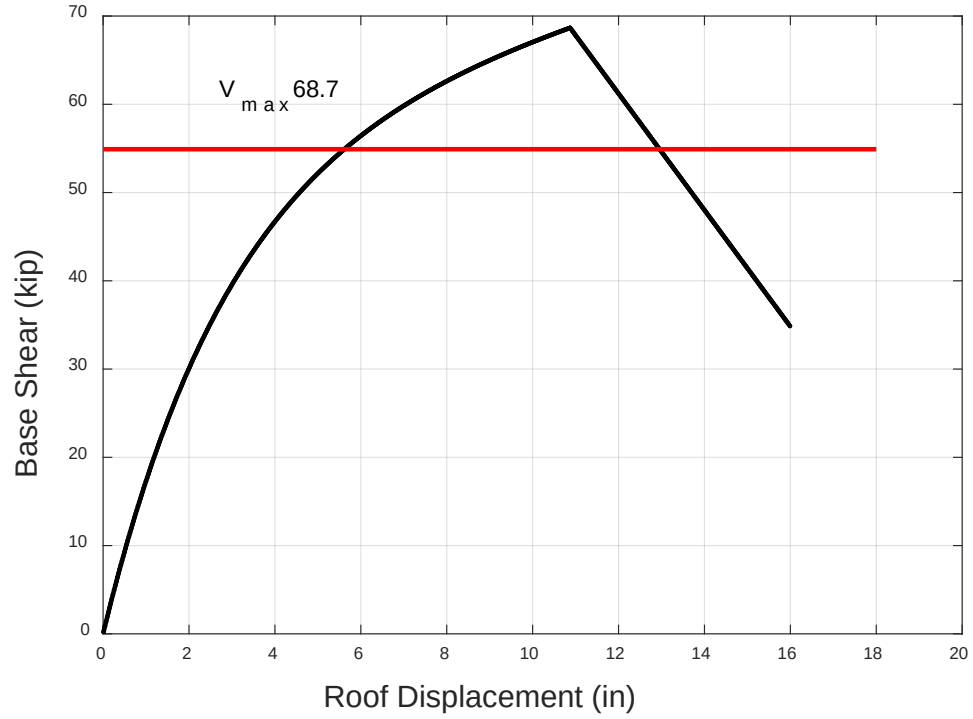


Figure I27. Archetype 26, R = 3, static pushover.

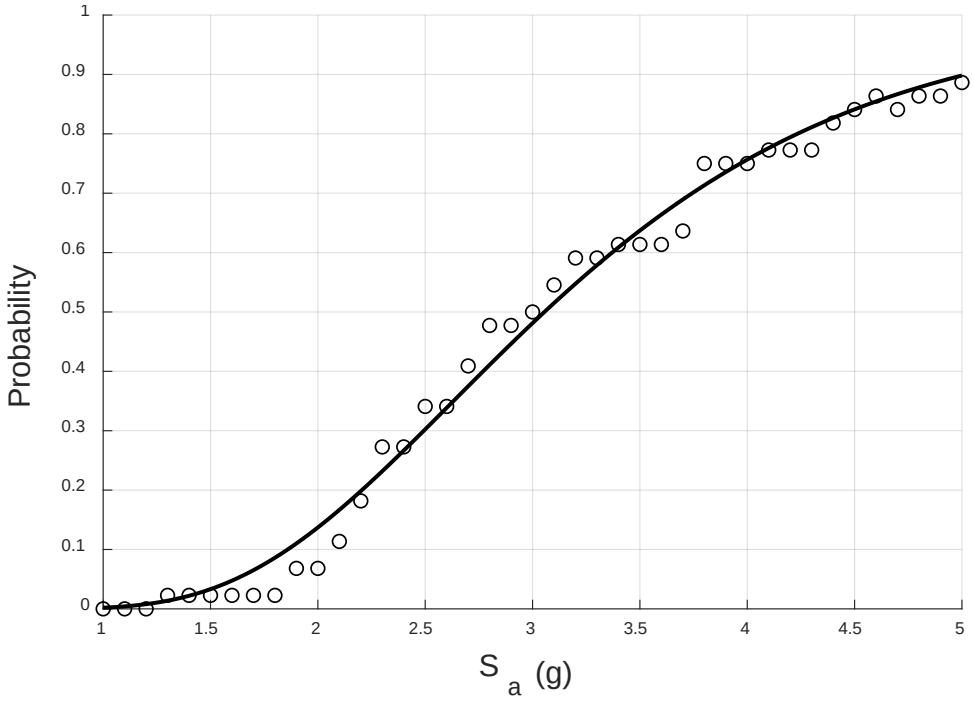


Figure I28. Archetype 26 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

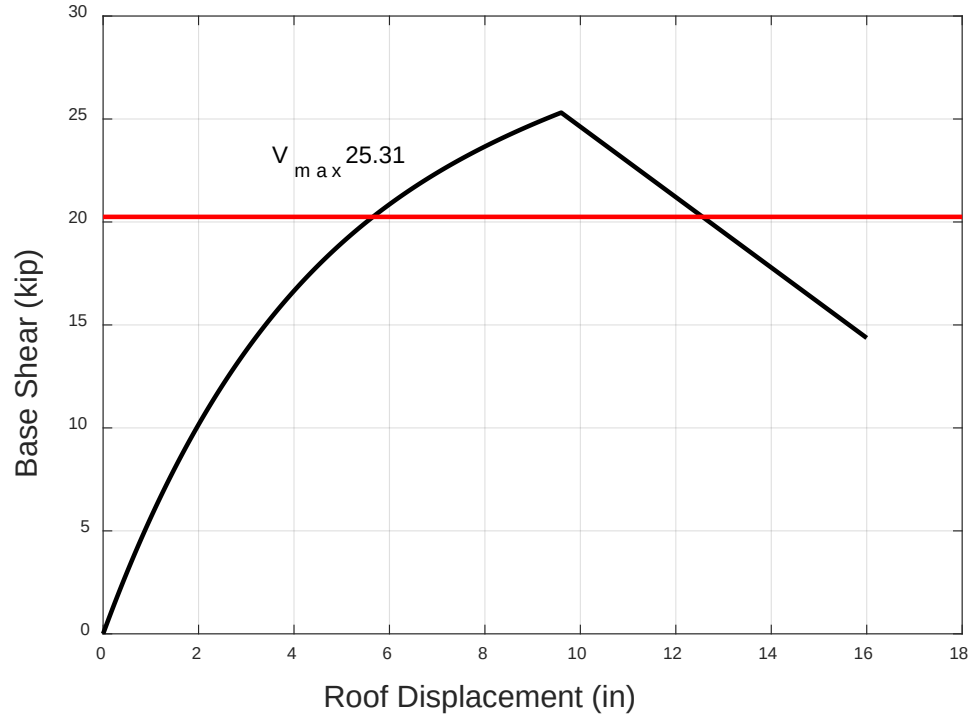


Figure I29. Archetype 27, R = 3, static pushover.

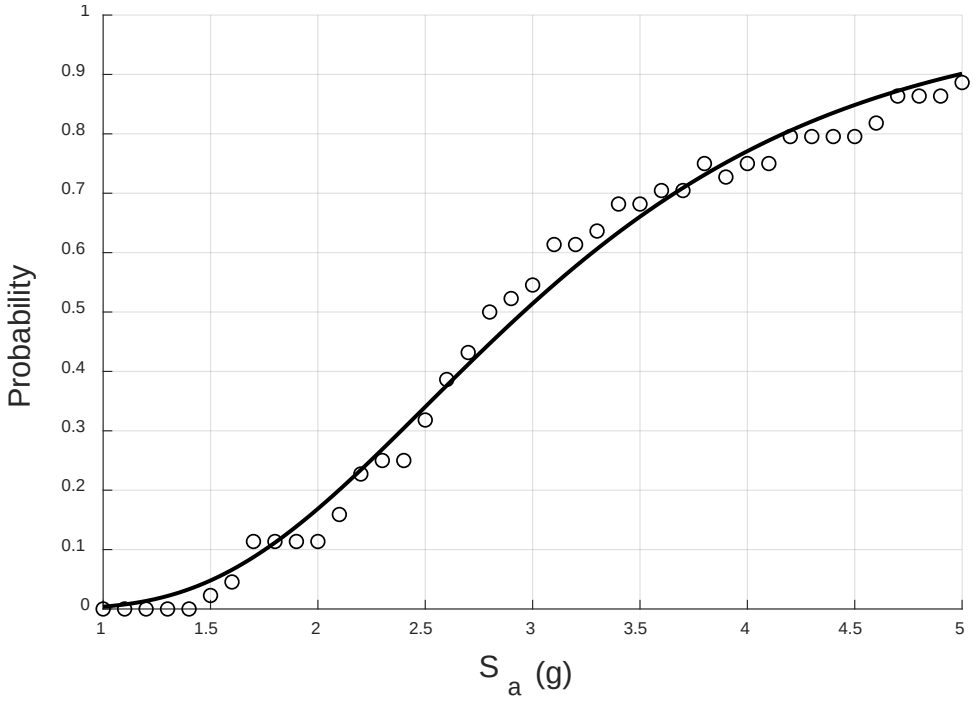


Figure I30. Archetype 27 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

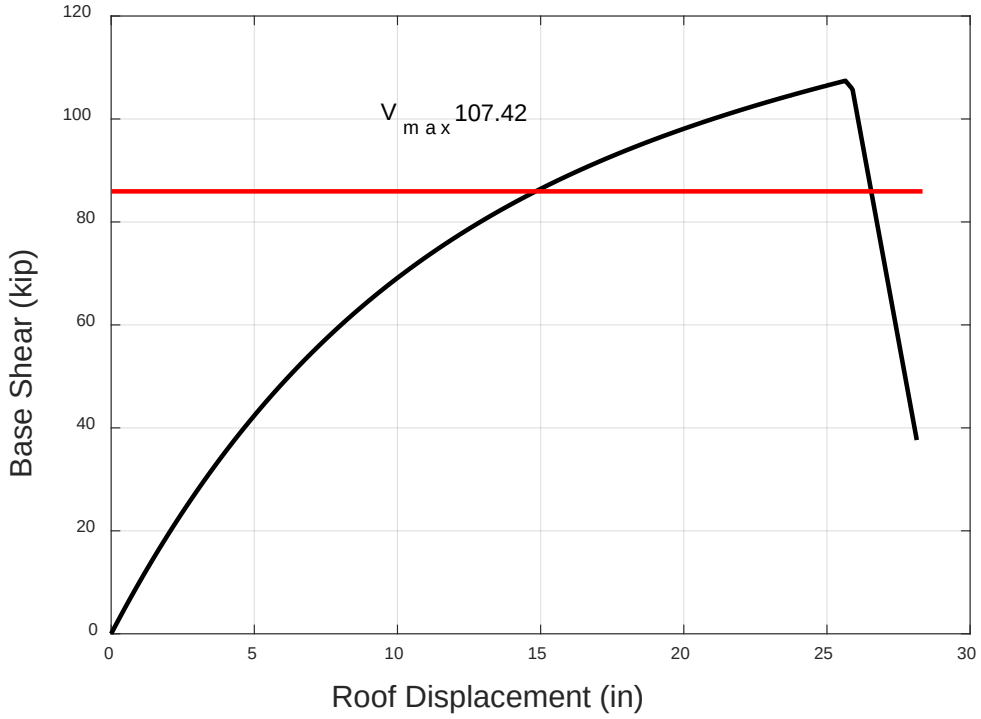


Figure I31. Archetype 28, R = 3, static pushover.

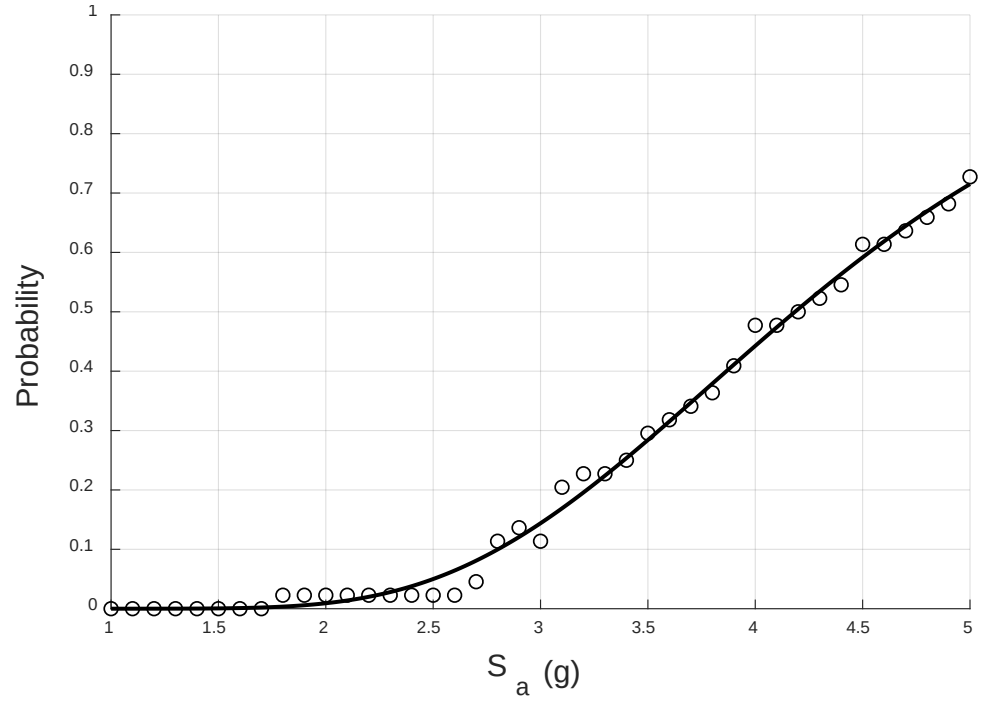


Figure I32. Archetype 28 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

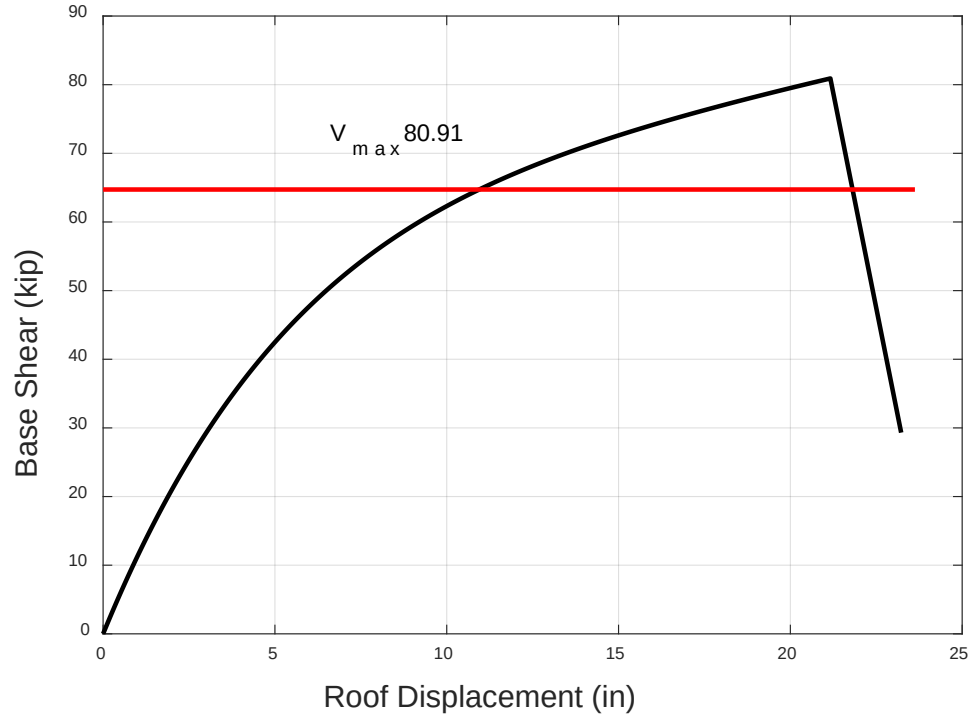


Figure I33. Archetype 29, R = 3, static pushover.

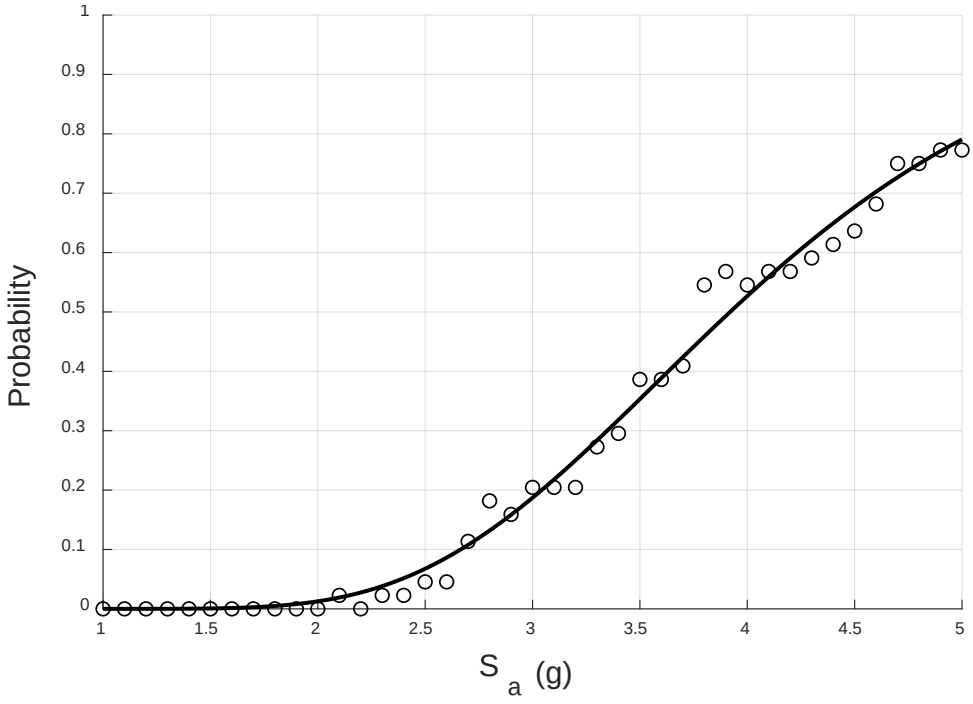


Figure I34. Archetype 29 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

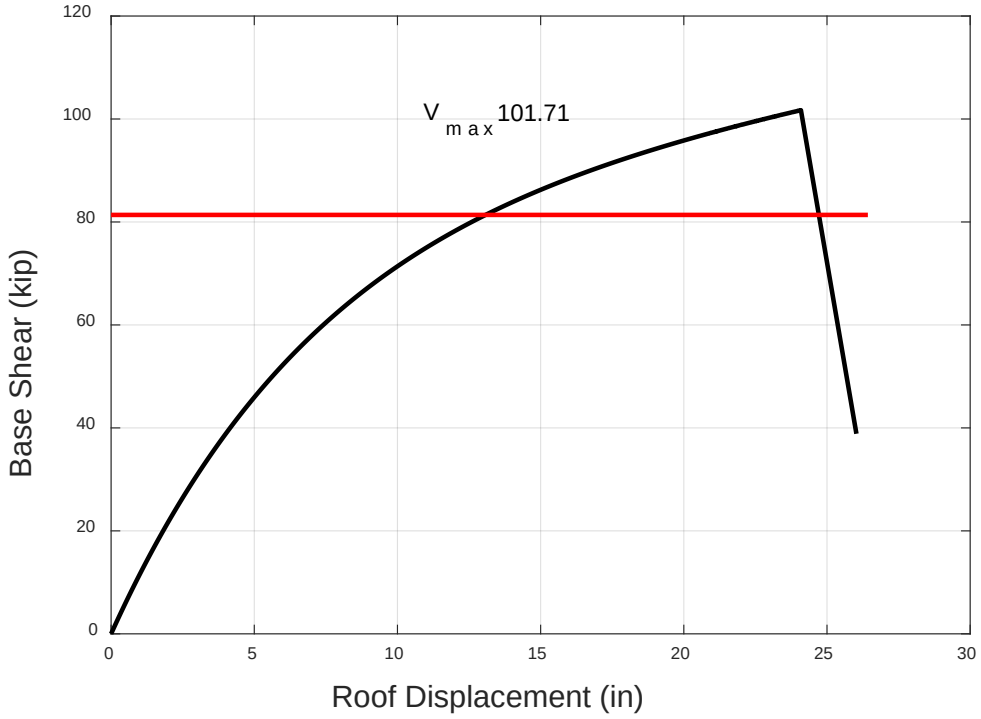


Figure I35. Archetype 30, R = 3, static pushover.

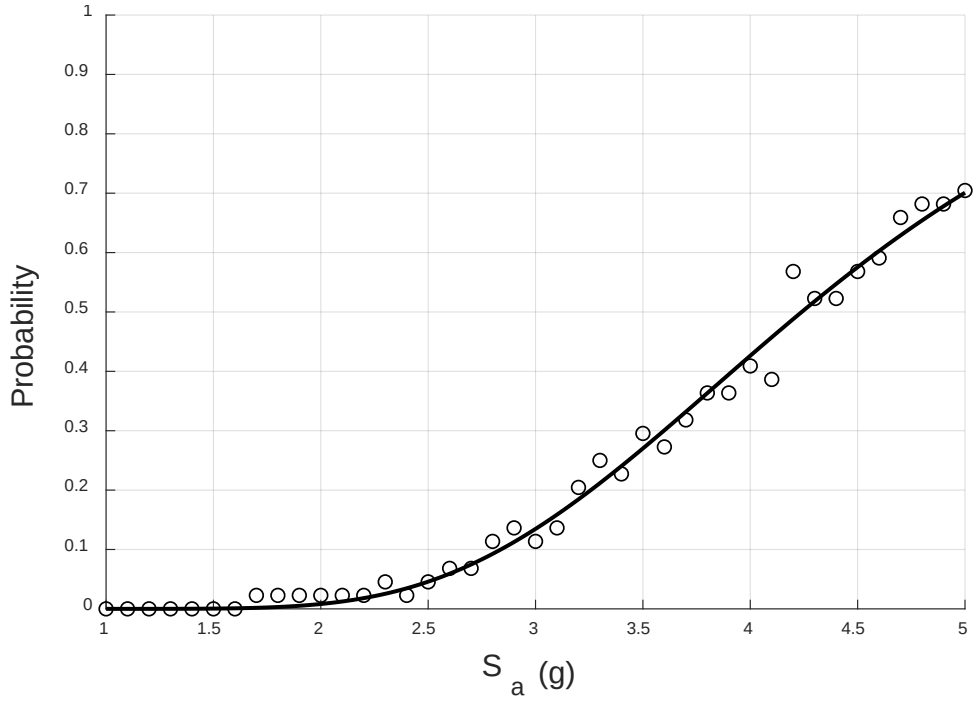


Figure I36. Archetype 30 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

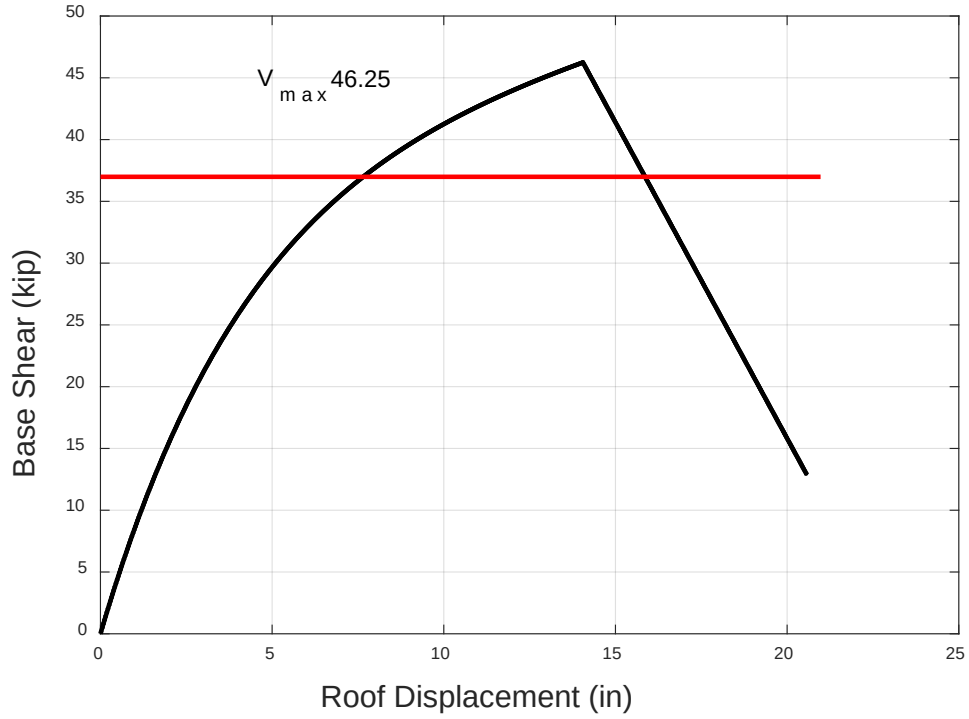


Figure I37. Archetype 37, R = 3, static pushover.

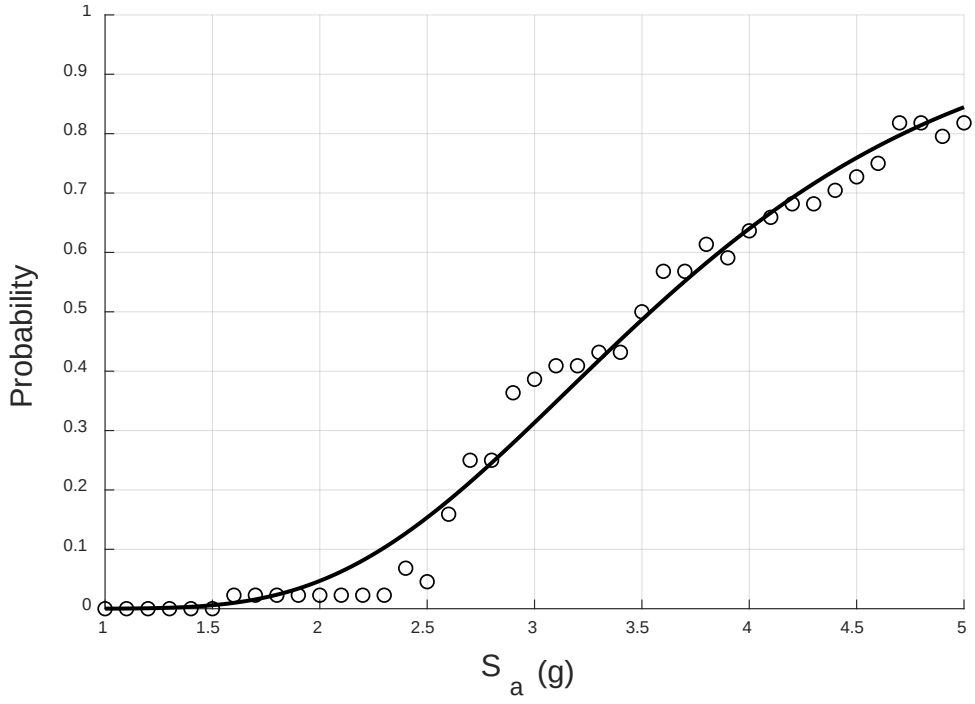


Figure I38. Archetype 37 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

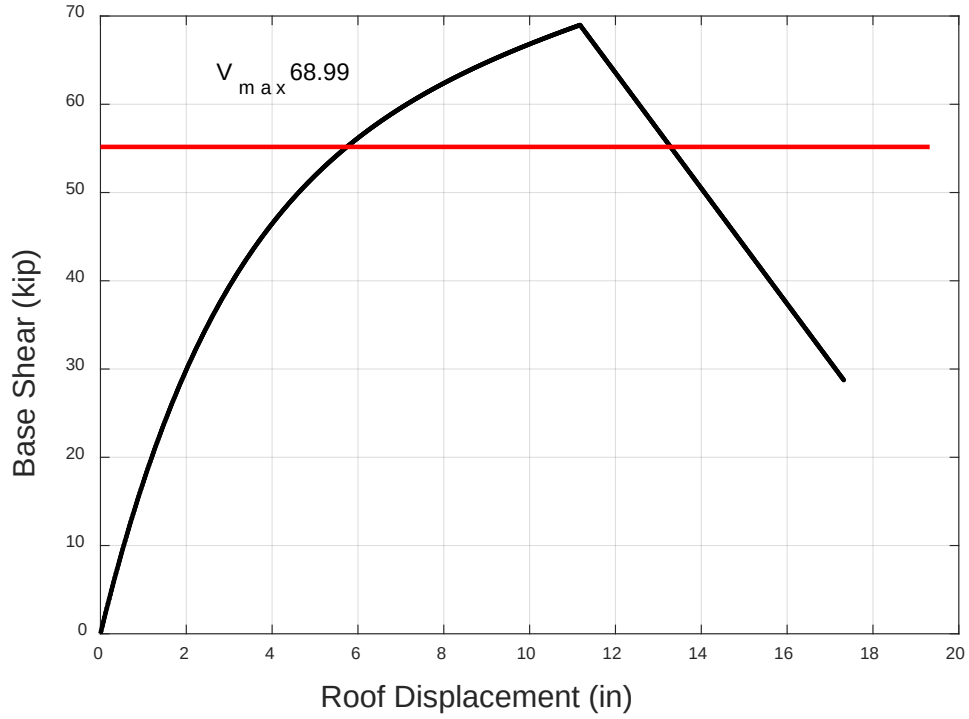


Figure I39. Archetype 38, R = 3, static pushover.

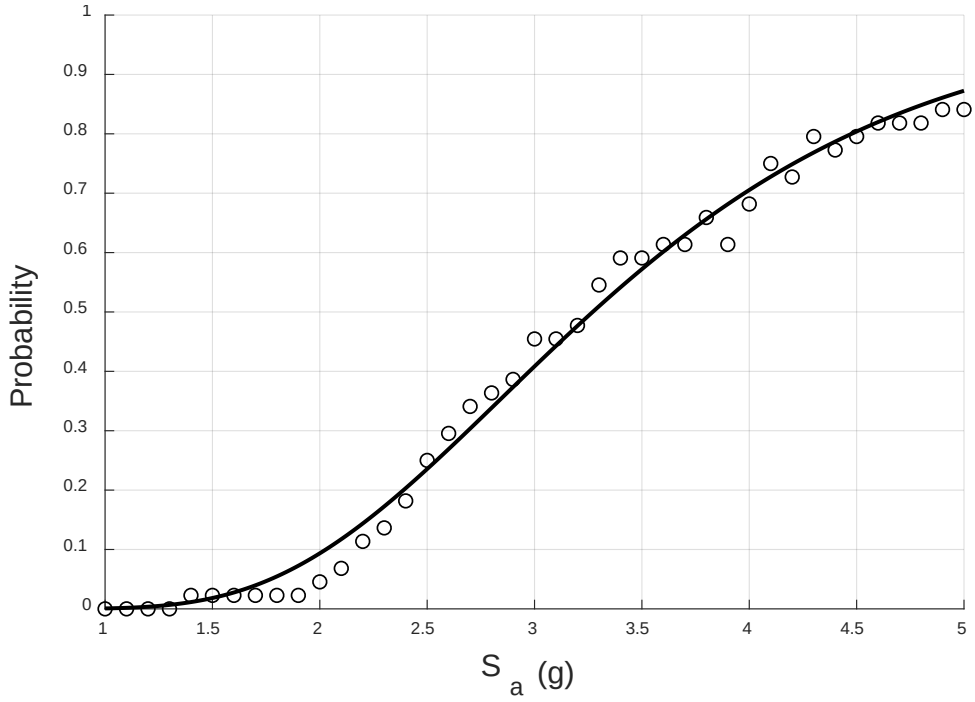


Figure I40. Archetype 38 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

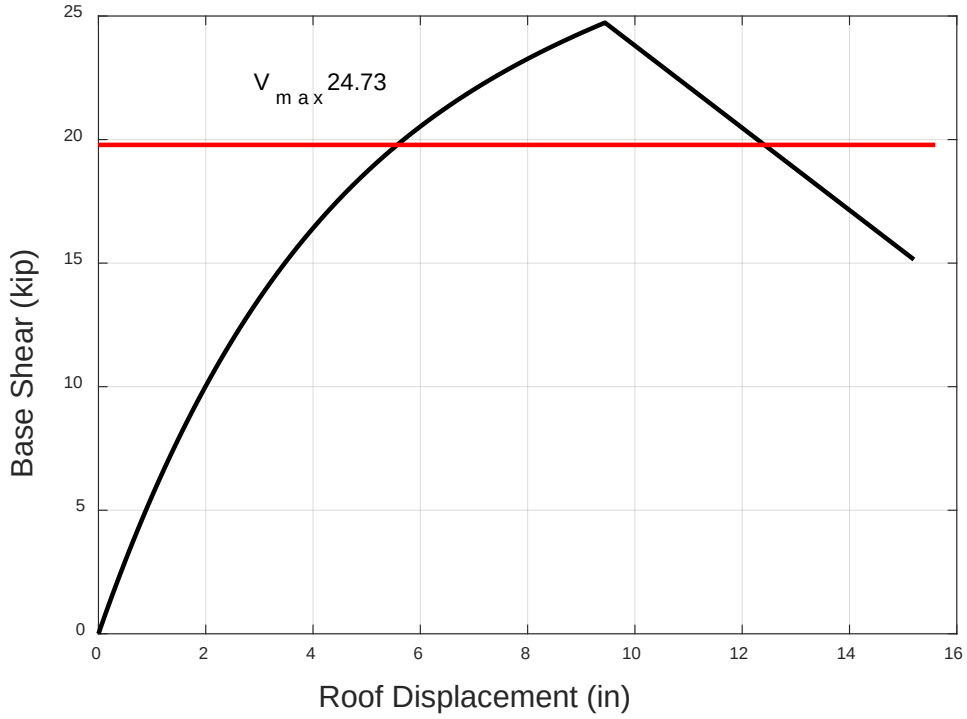


Figure I41. Archetype 39, R = 3, static pushover.

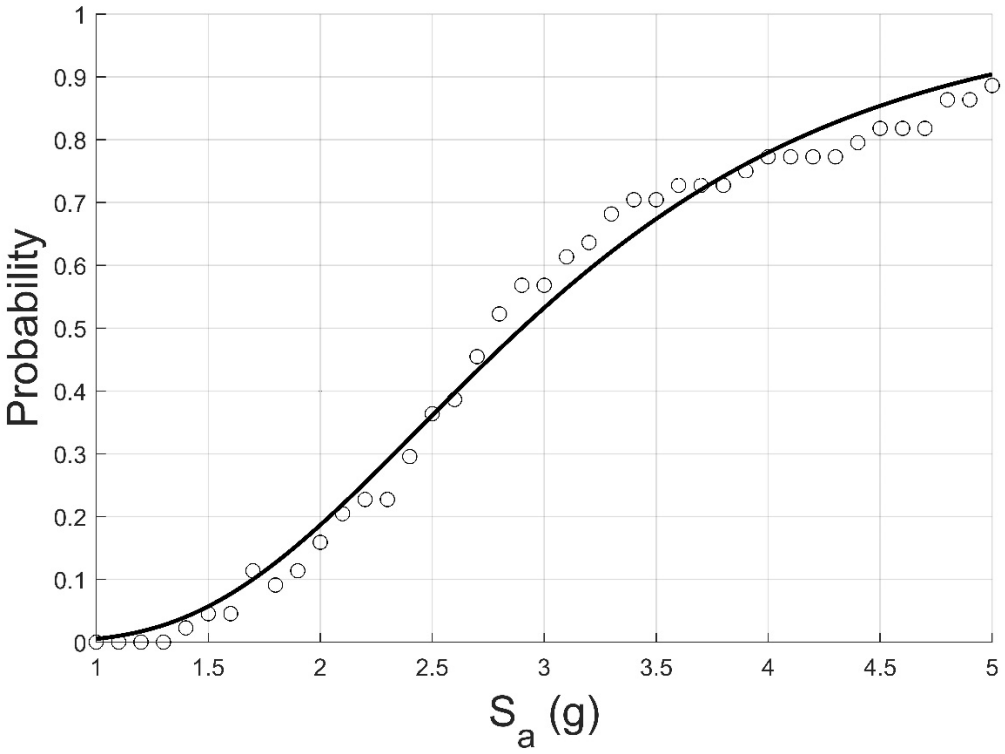


Figure I42. Archetype 39 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

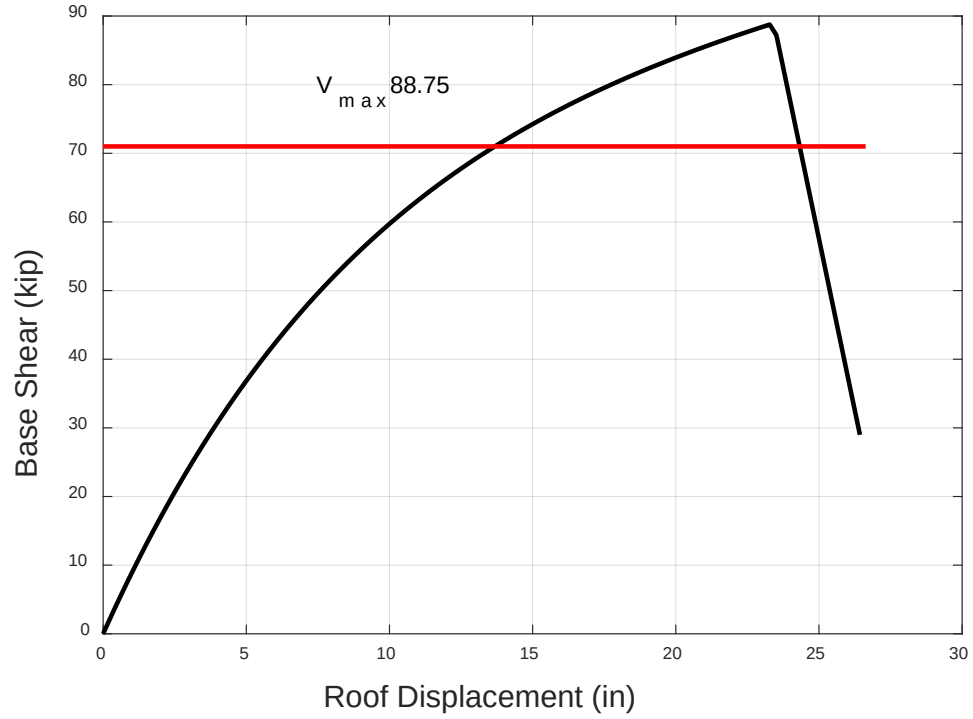


Figure I43. Archetype 40, R = 3, static pushover.

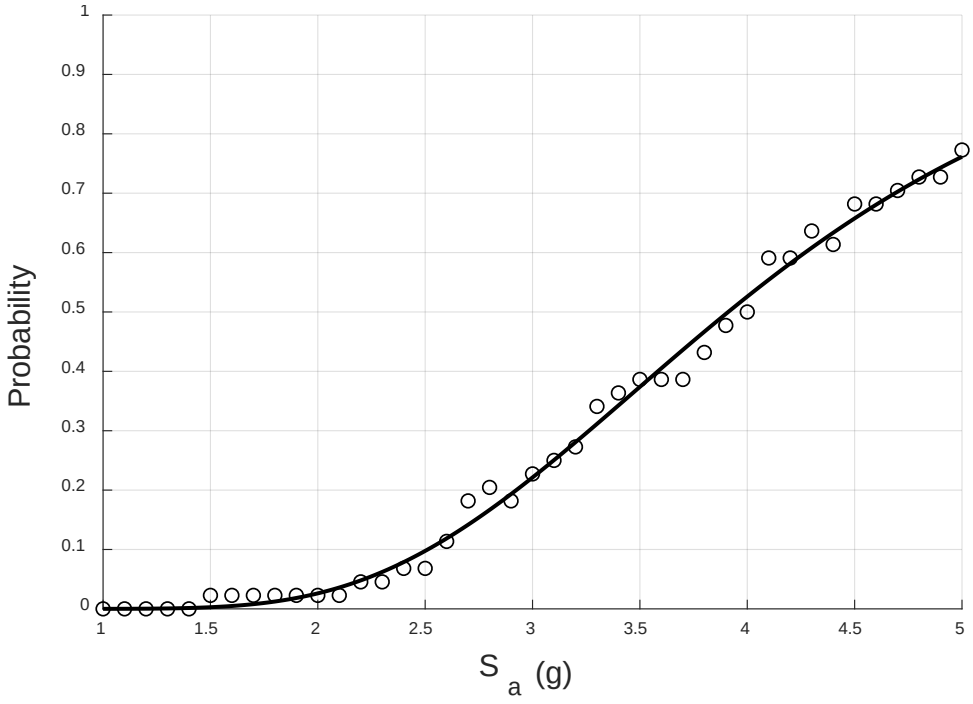


Figure I44. Archetype 40 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

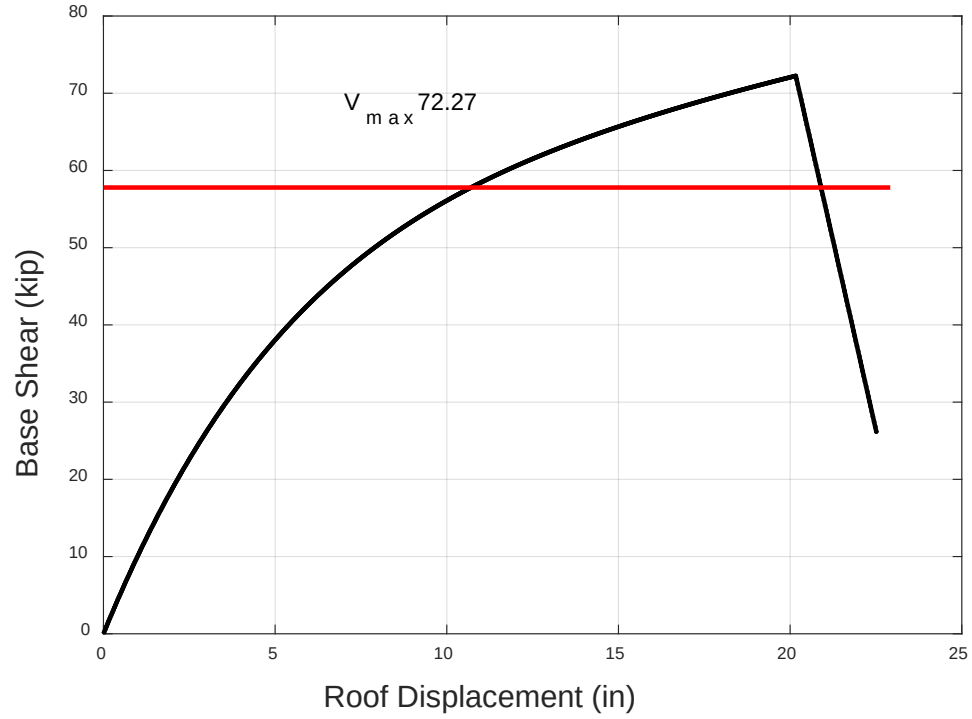


Figure I45. Archetype 41, R = 3, static pushover.

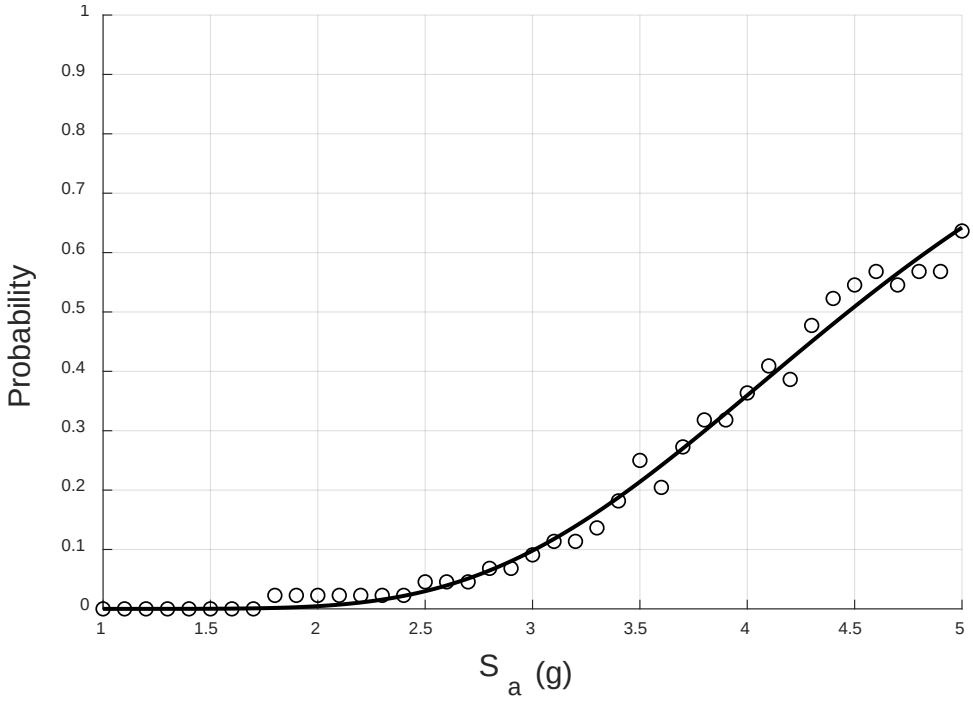


Figure I46. Archetype 41 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

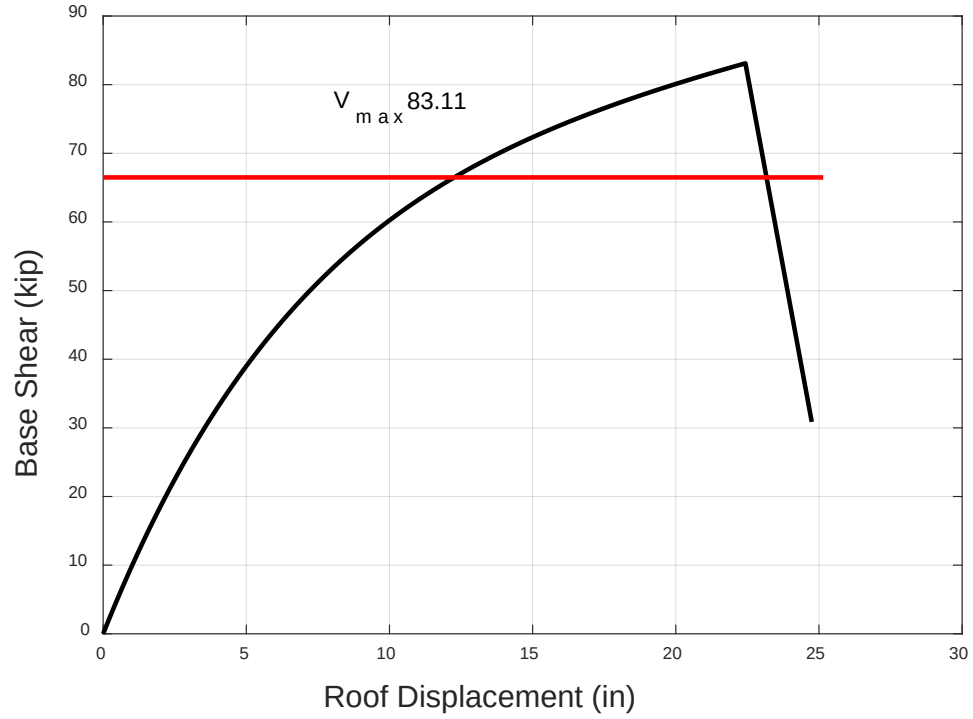


Figure I47. Archetype 42, R = 3, static pushover.

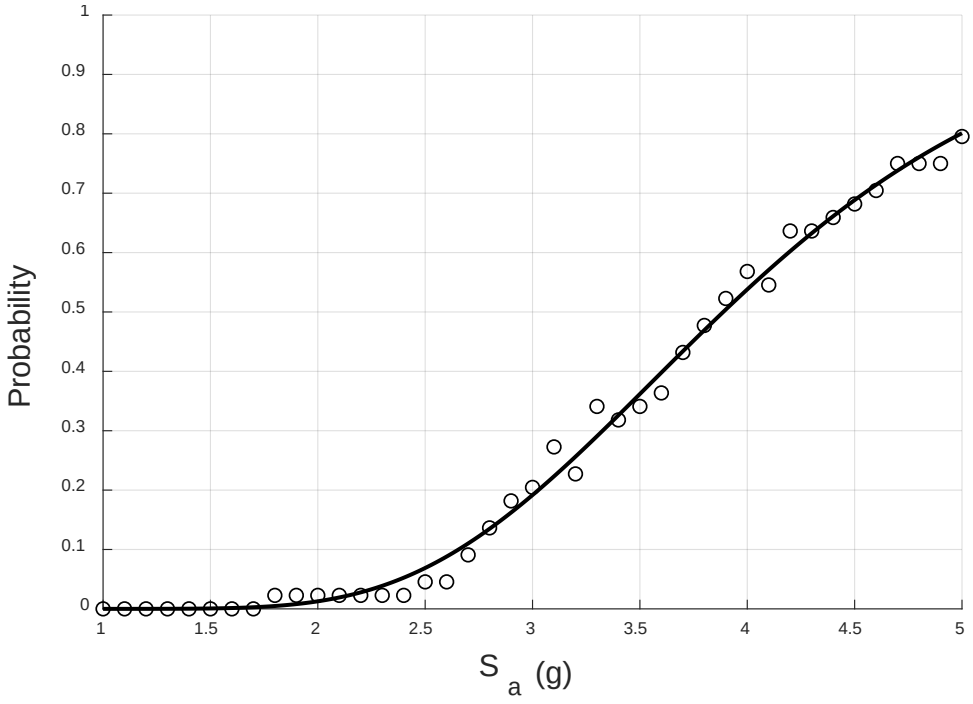


Figure I48. Archetype 42 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

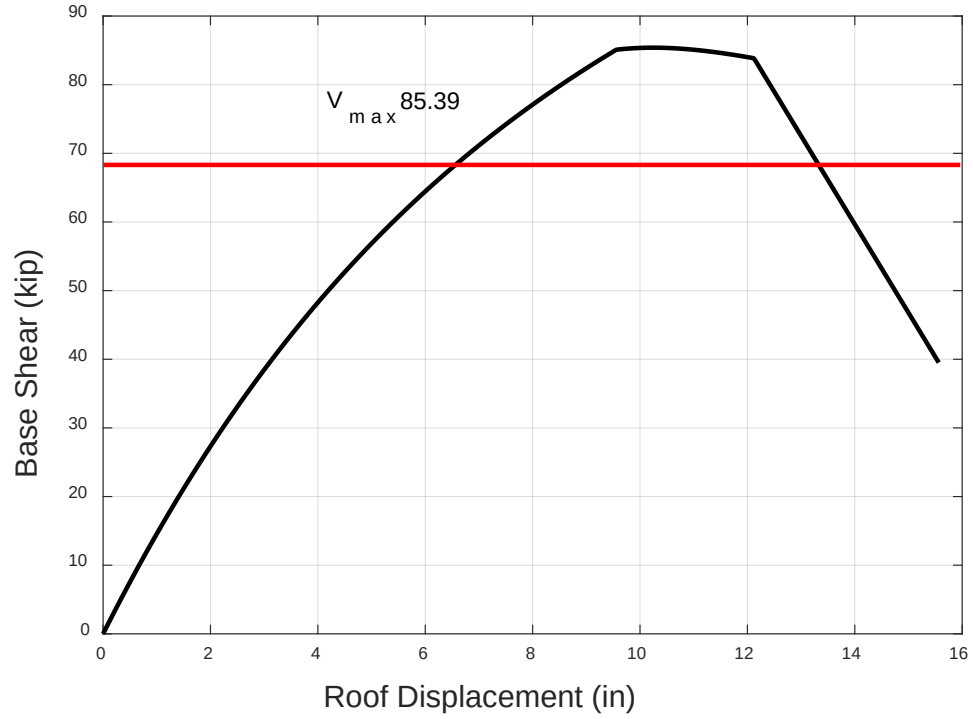


Figure I49. Archetype 49, R = 3, static pushover.

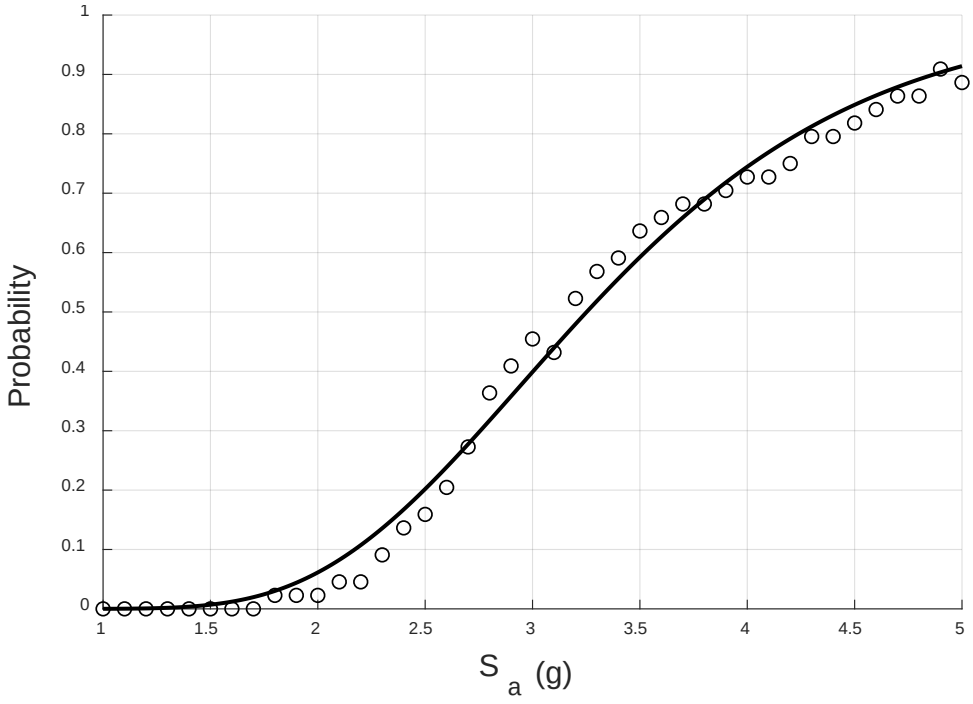


Figure I50. Archetype 49 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

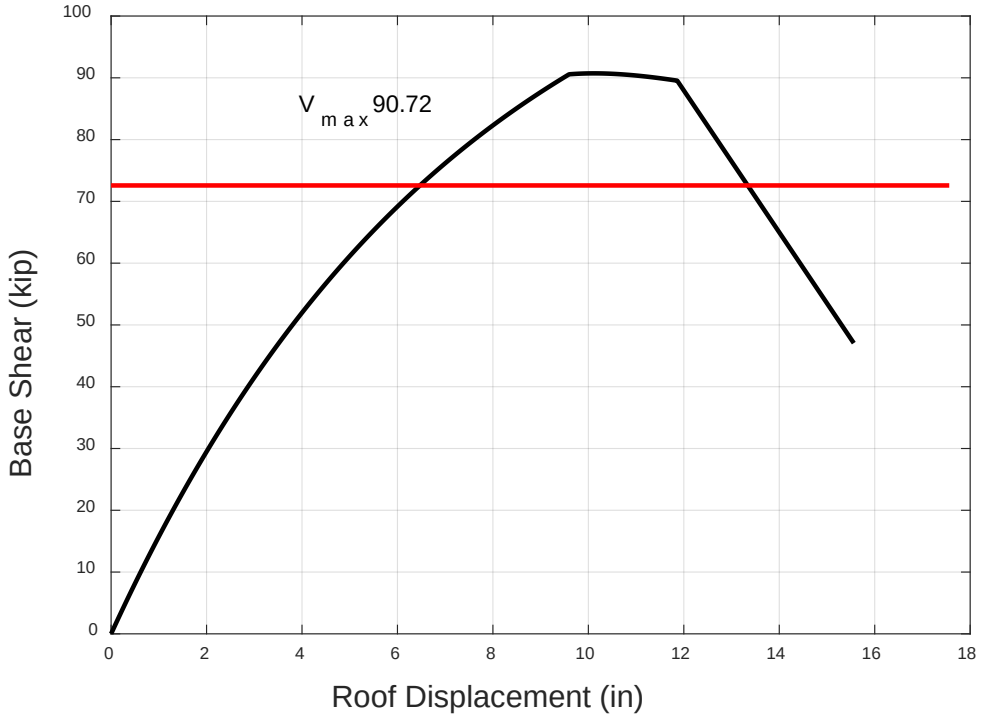


Figure I51. Archetype 50, R = 3, static pushover.

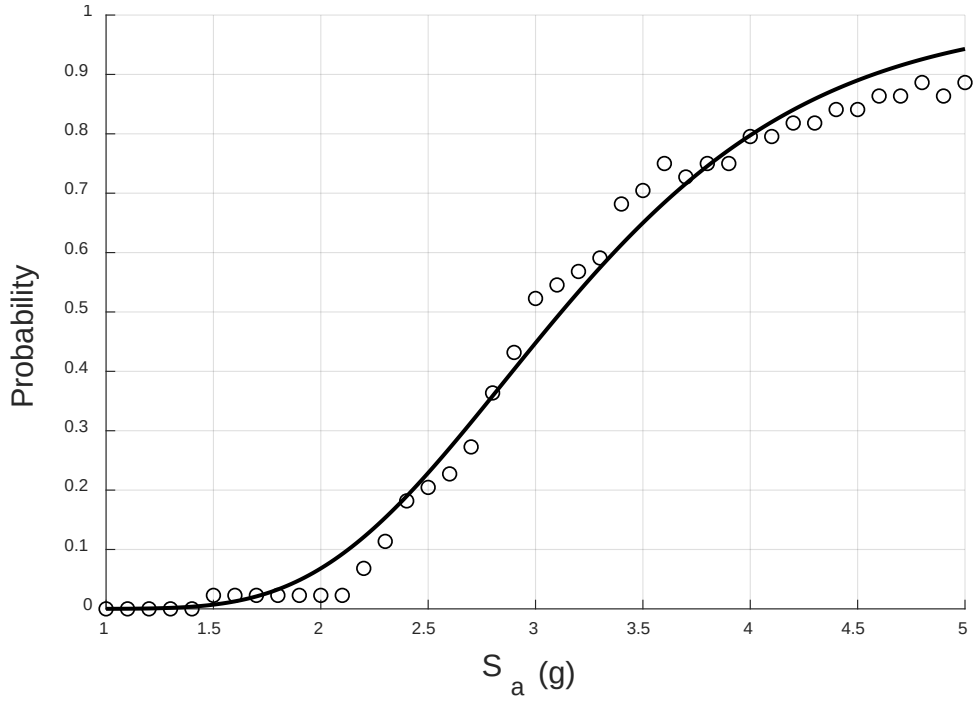


Figure I52. Archetype 50 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

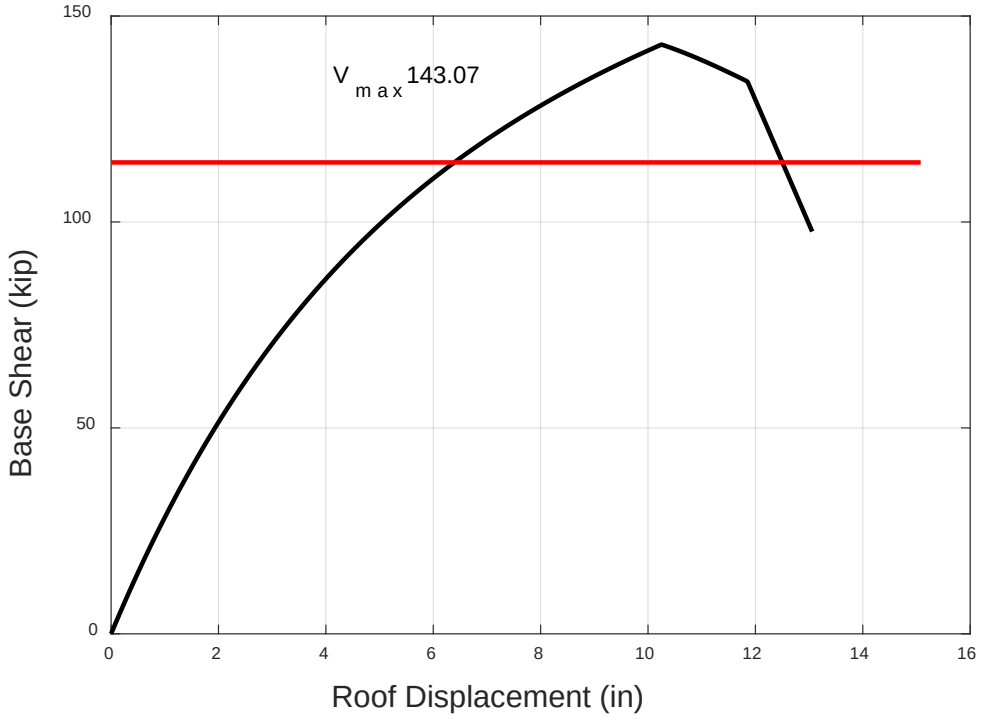


Figure I53. Archetype 51, R = 3, static pushover.

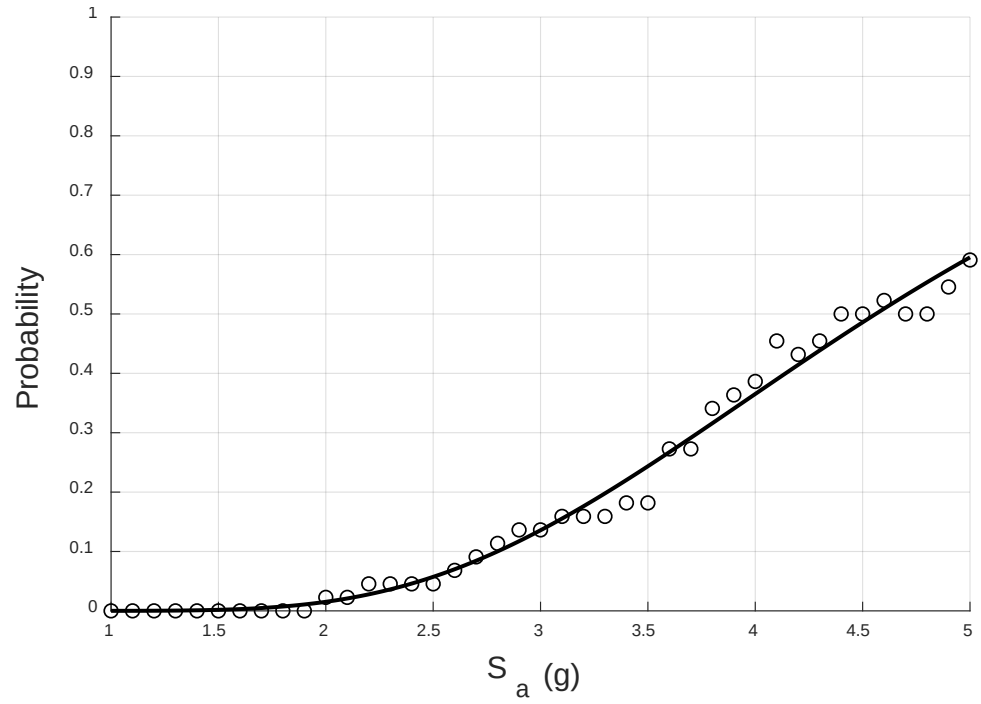


Figure I54. Archetype 51 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

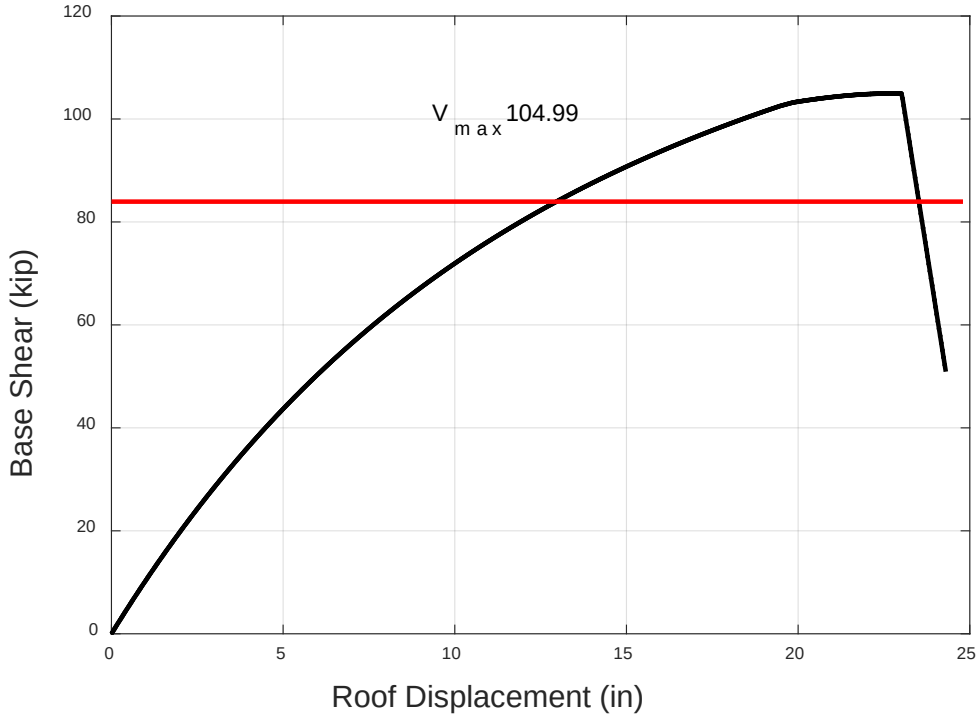


Figure I55. Archetype 52, R = 3, static pushover.

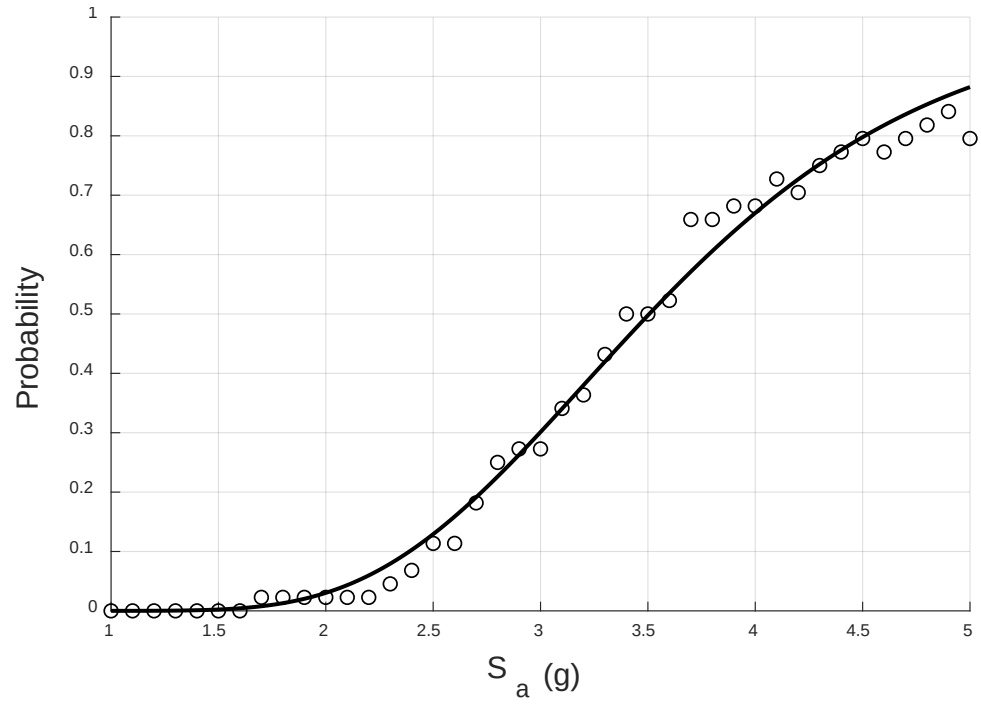


Figure I56. Archetype 52 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

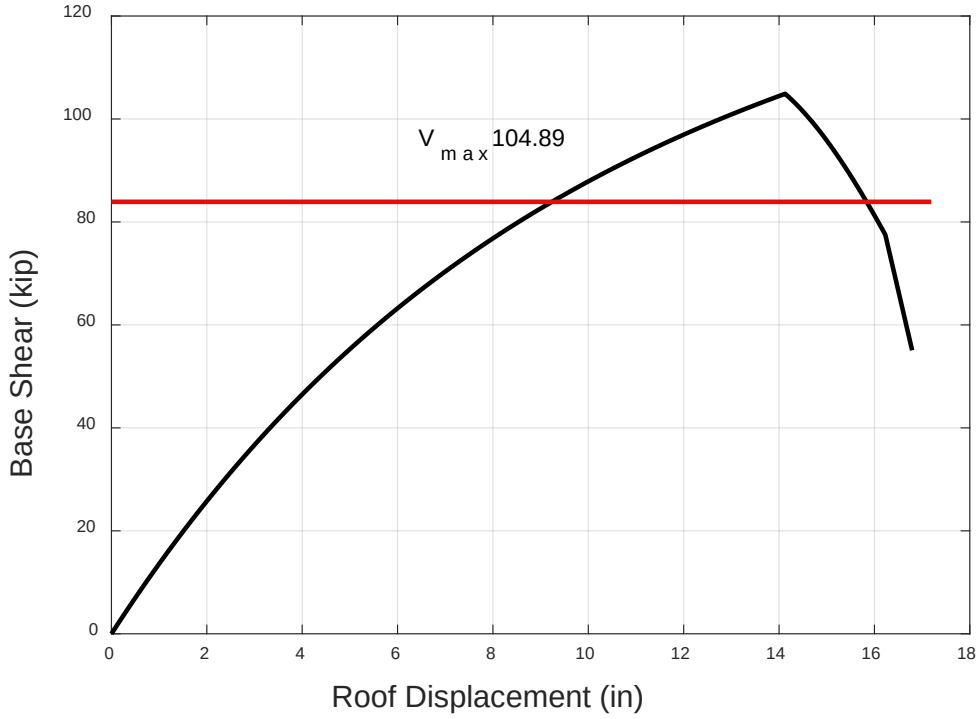


Figure I57. Archetype 53, R = 3, static pushover.

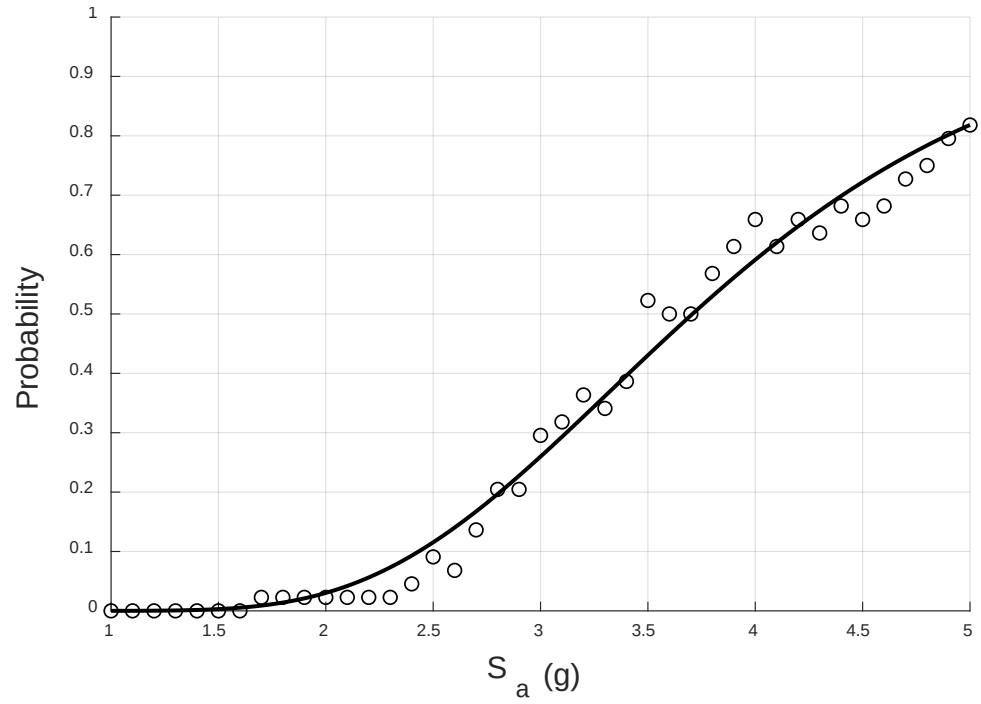


Figure I58. Archetype 53 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

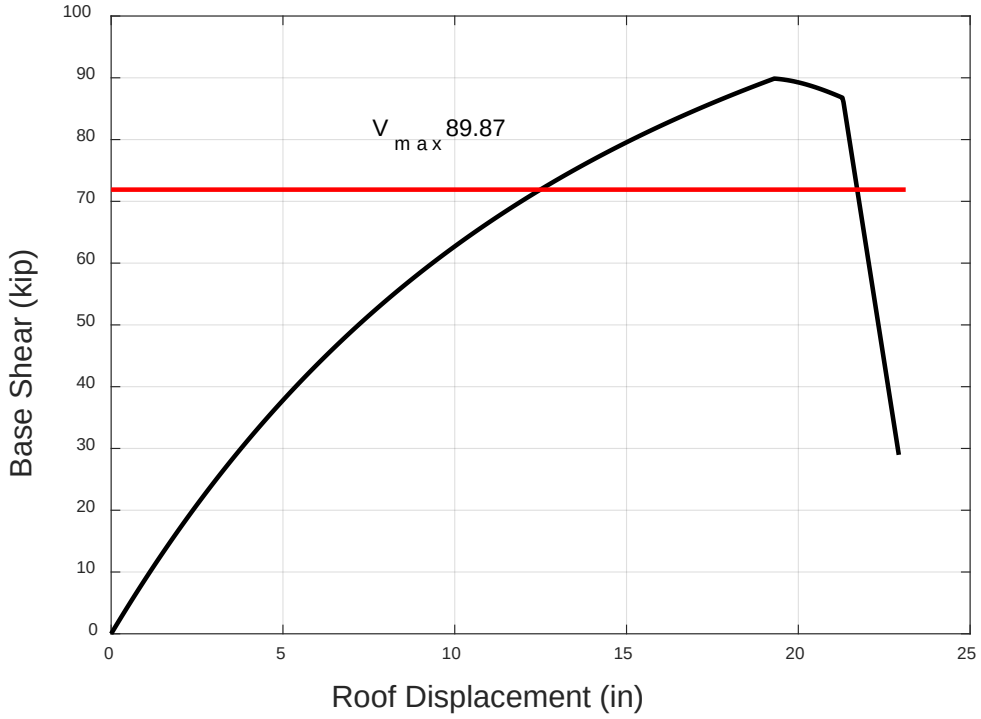


Figure I59. Archetype 54, R = 3, static pushover.

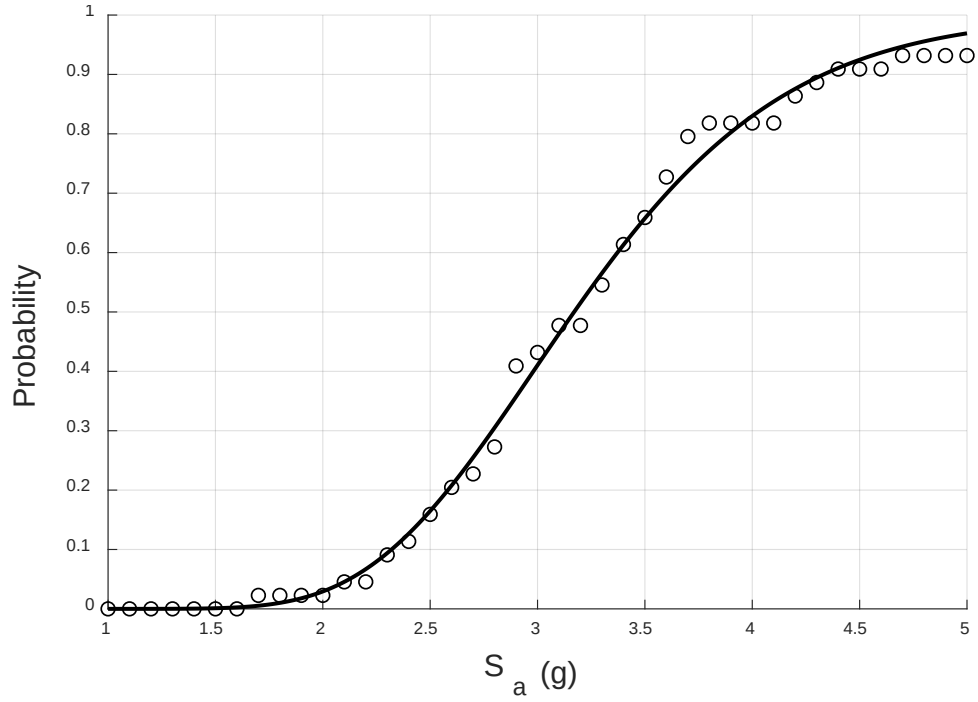


Figure I60. Archetype 54 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

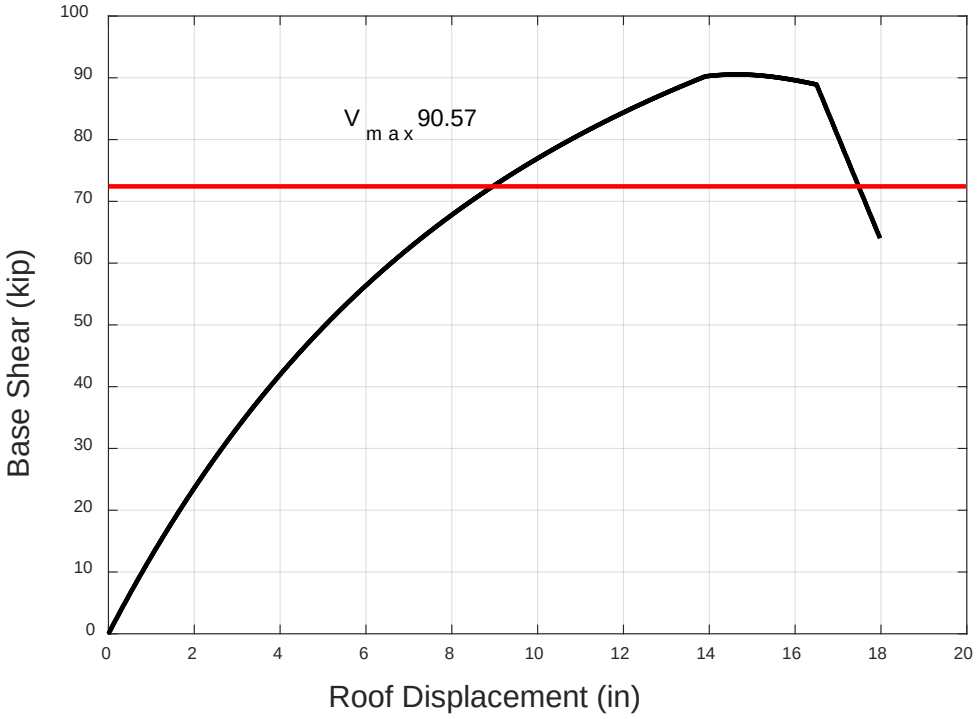


Figure I61. Archetype 61, R = 3, static pushover.

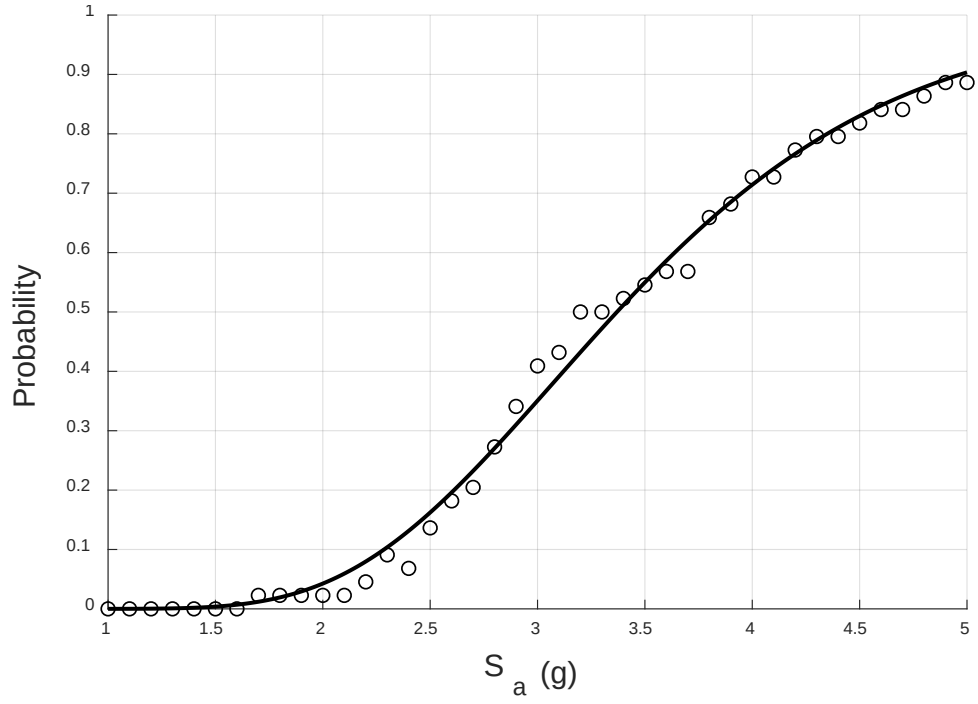


Figure I62. Archetype 61 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

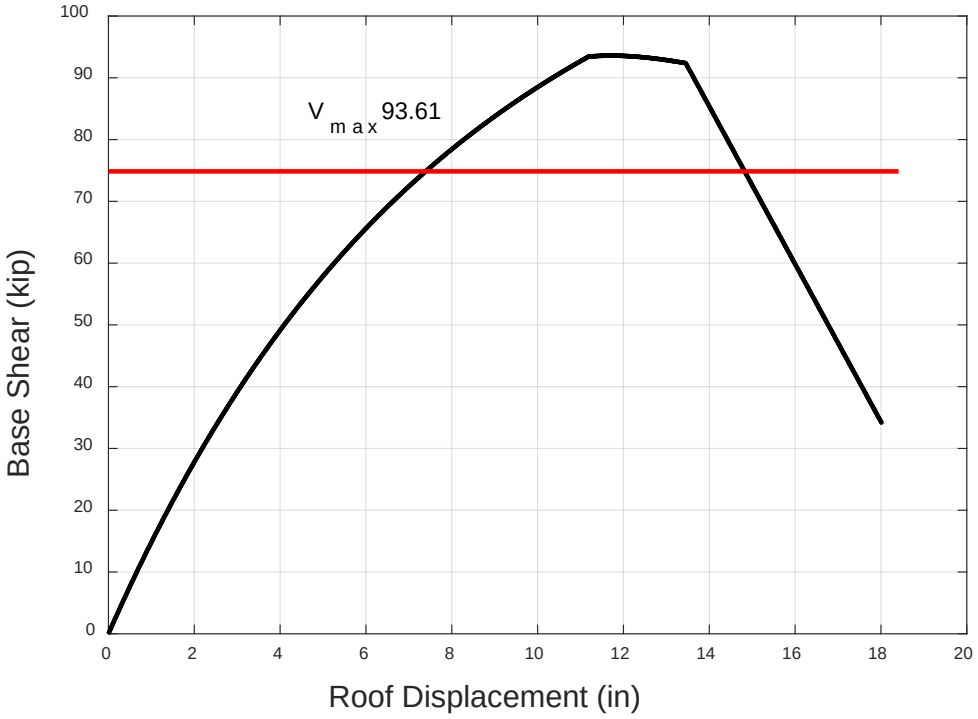


Figure I63. Archetype 62, R = 3, static pushover.

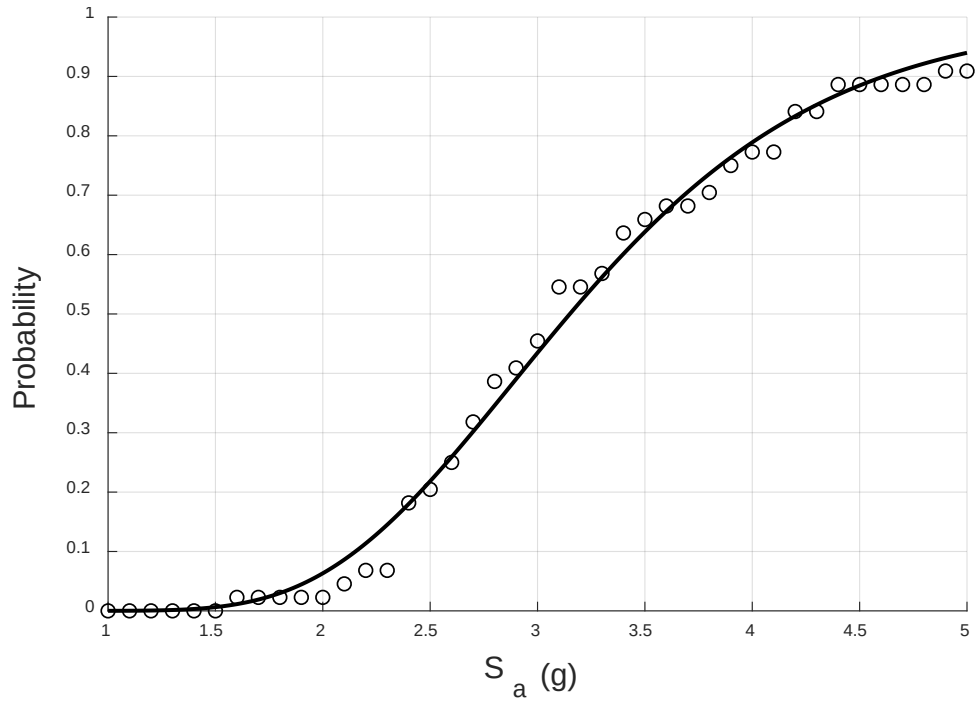


Figure I64. Archetype 62 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

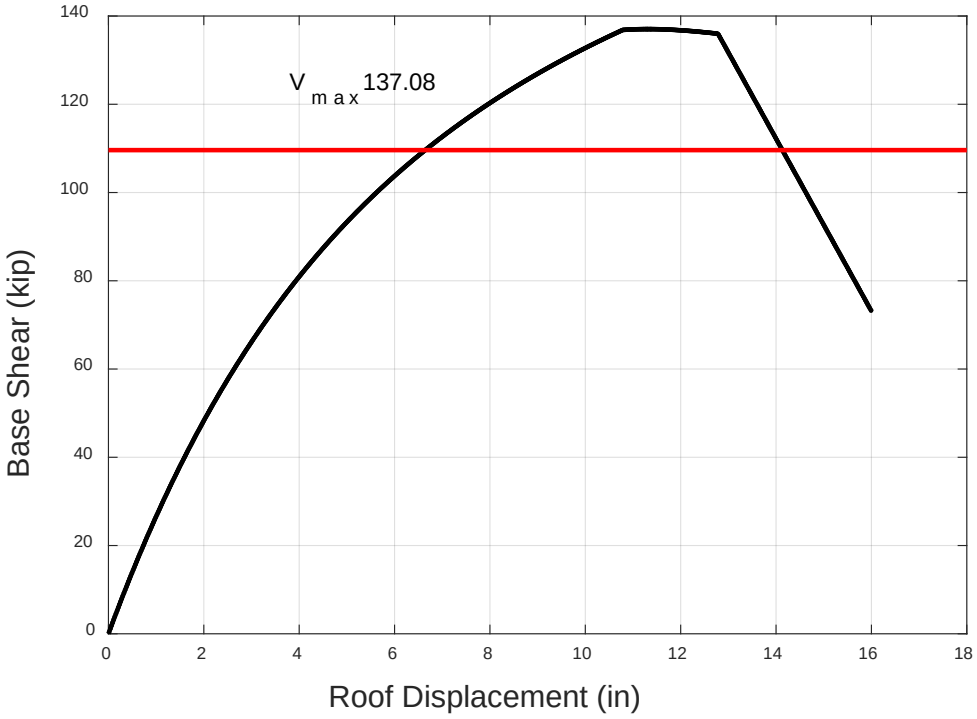


Figure I65. Archetype 63, R = 3, static pushover.

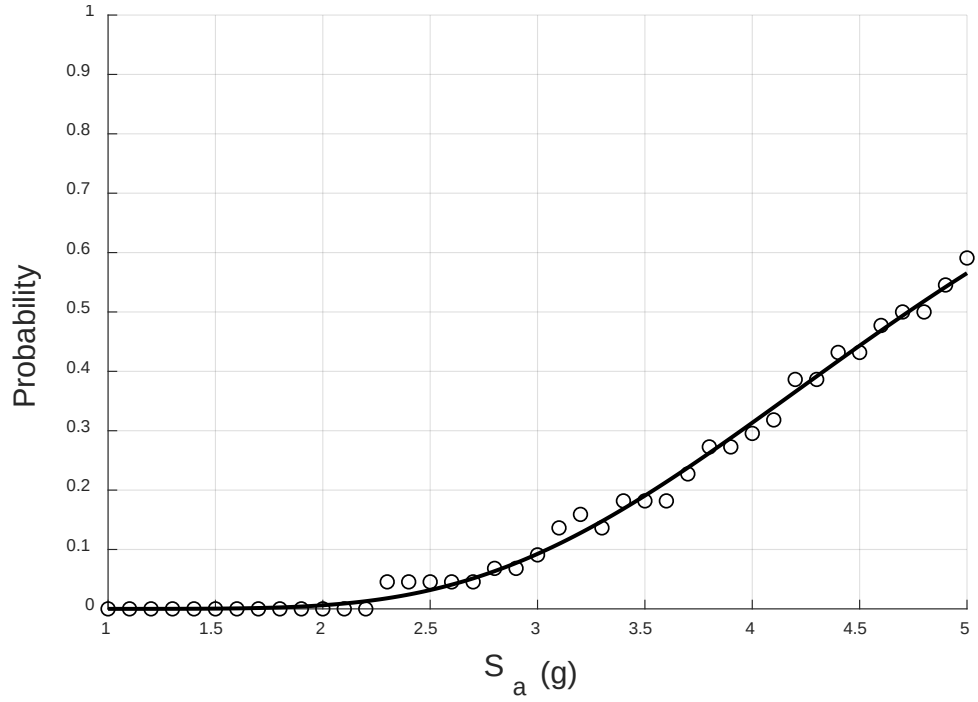


Figure I66. Archetype 63 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

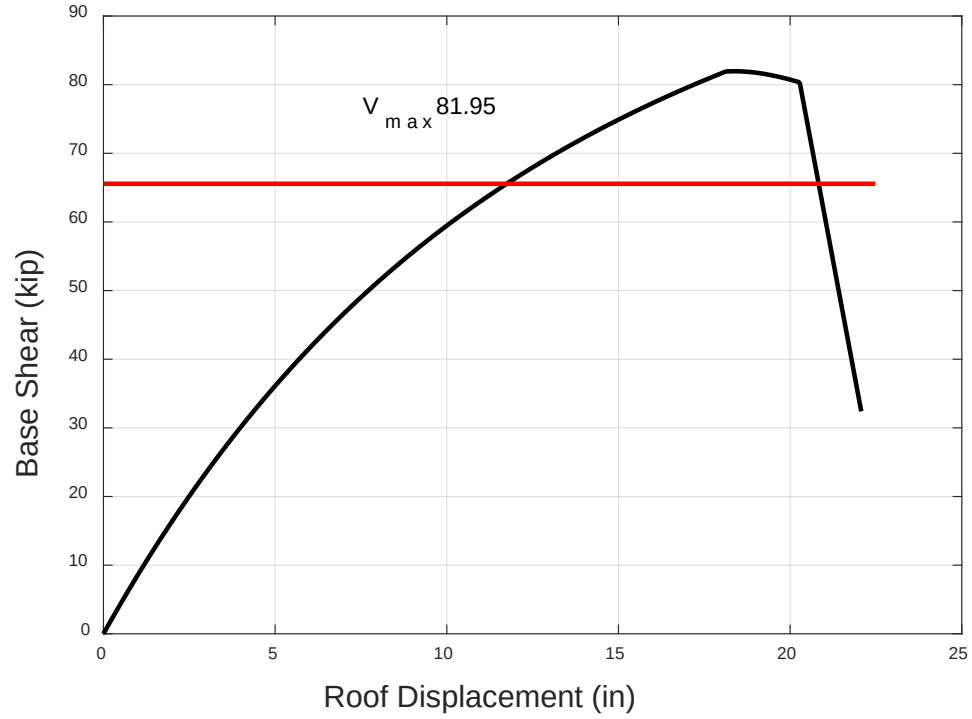


Figure I67. Archetype 64, R = 3, static pushover.

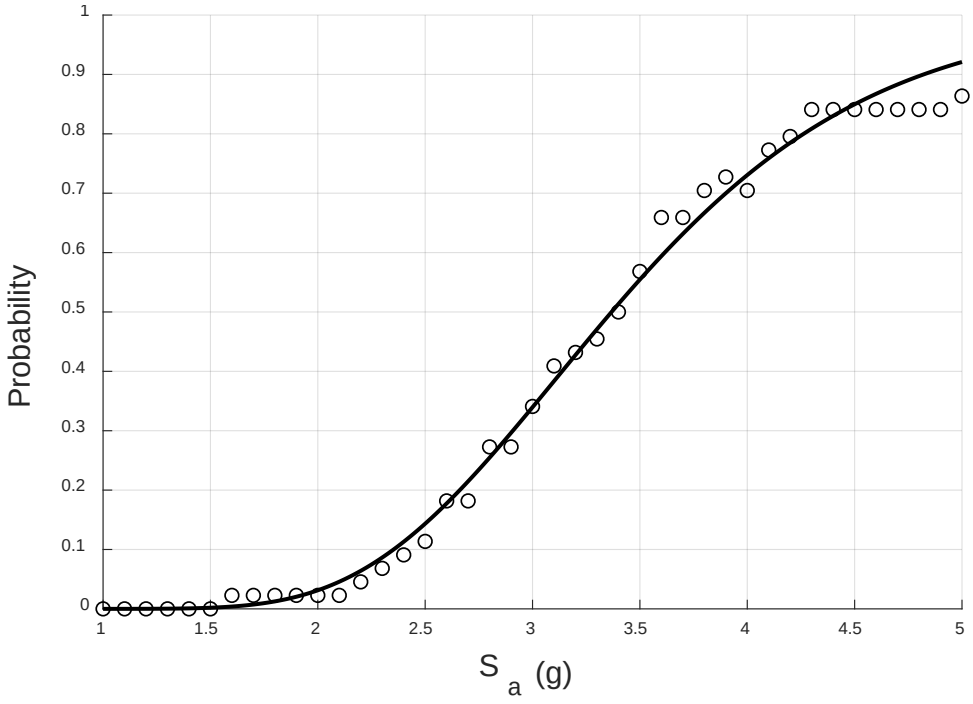


Figure I68. Archetype 64 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

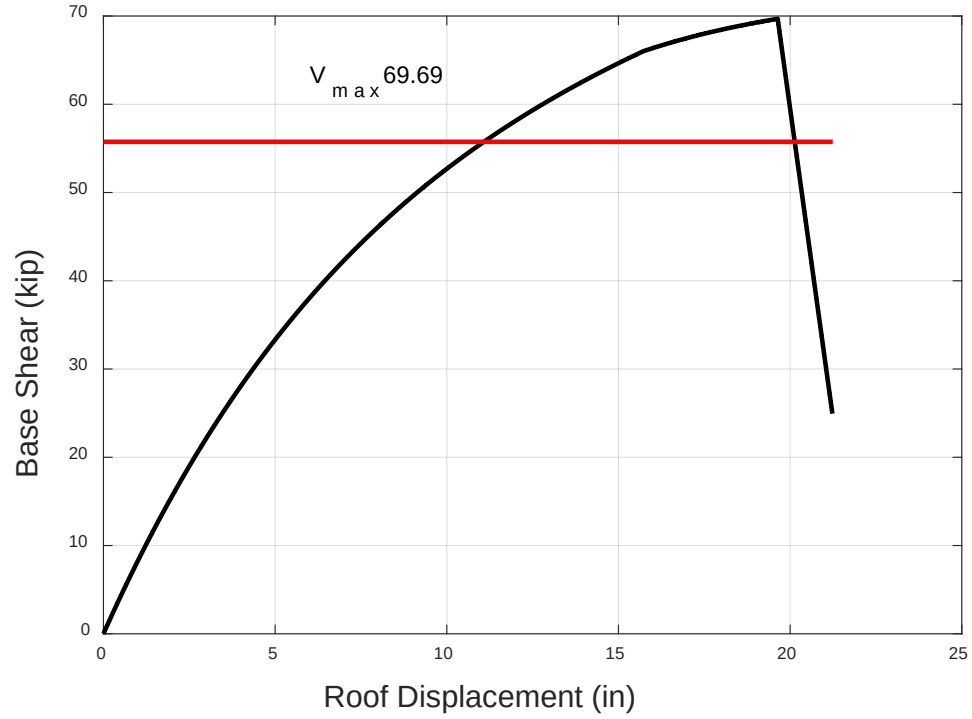


Figure I69. Archetype 65, R = 3, static pushover.

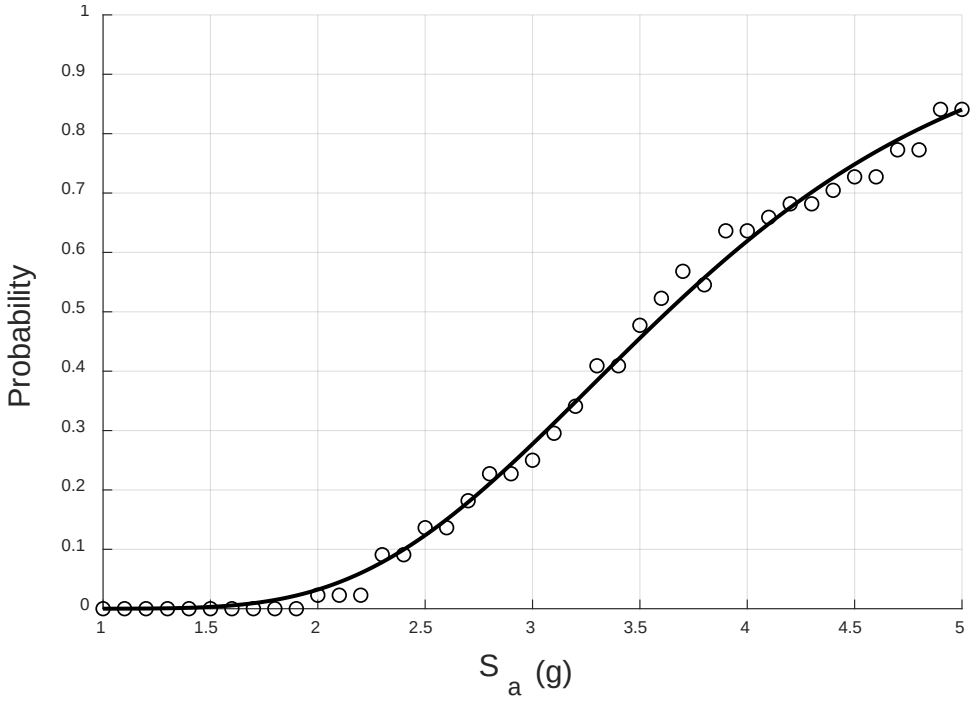


Figure I70. Archetype 65 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

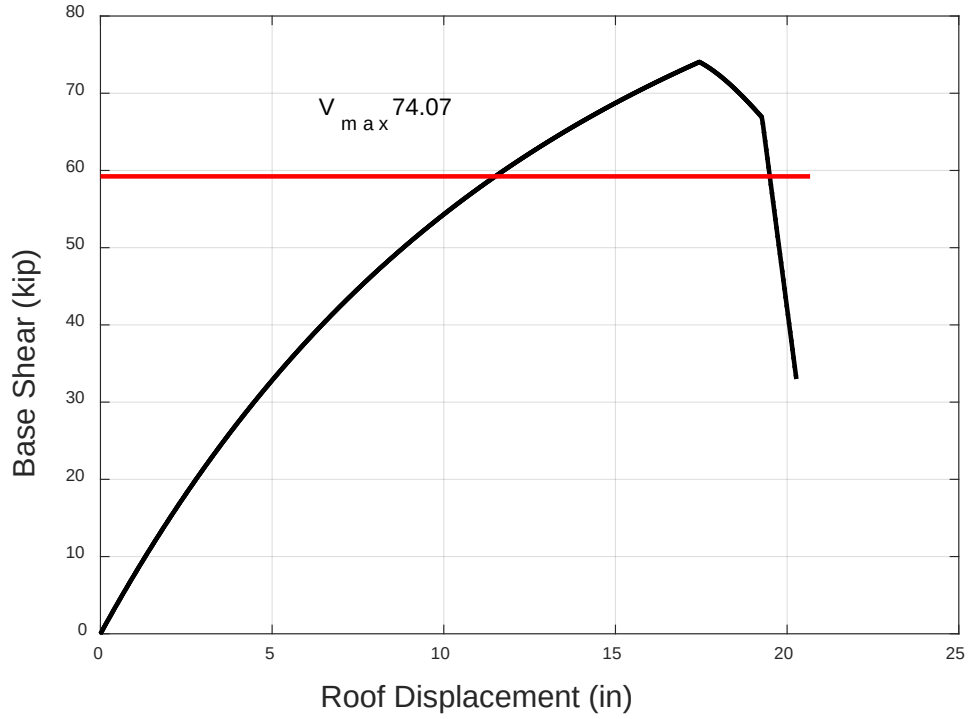


Figure I71. Archetype 66, R = 3, static pushover.

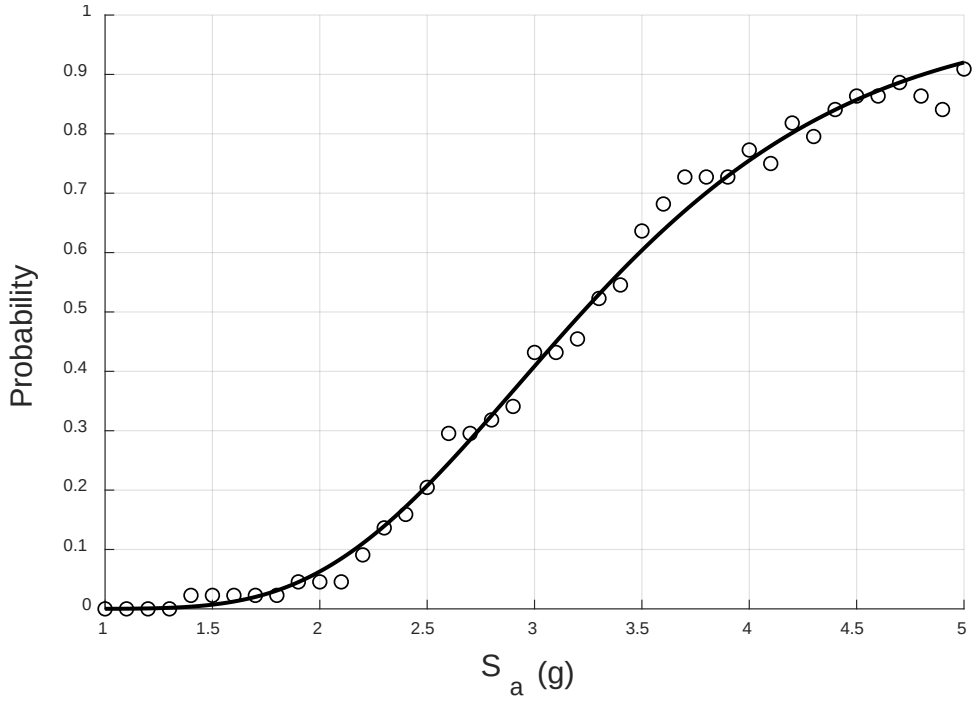


Figure I72. Archetype 66 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

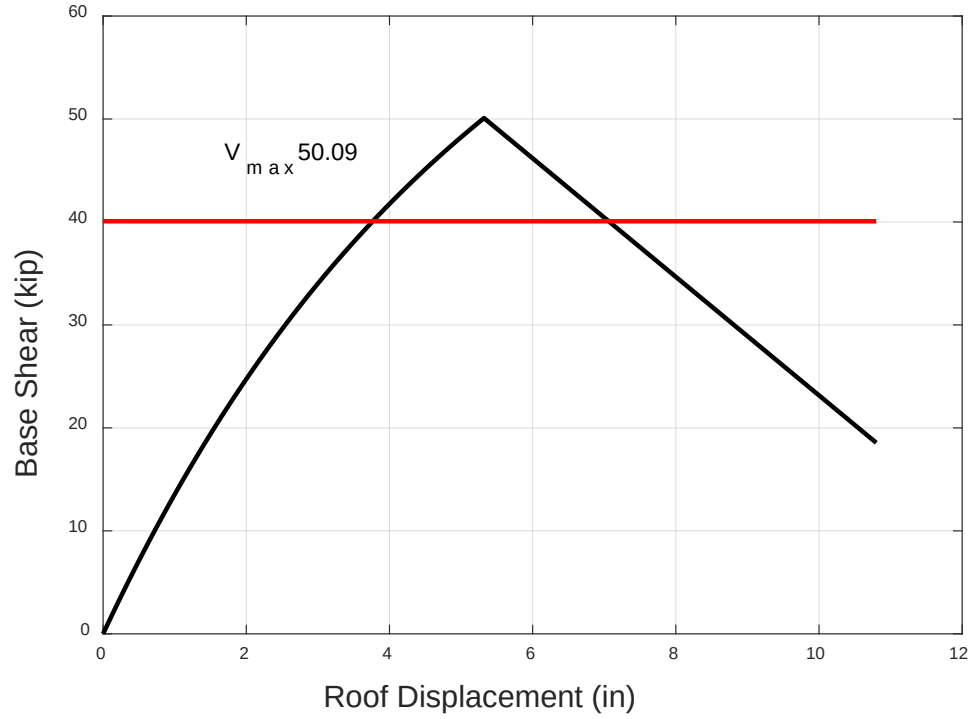


Figure I73. Archetype 73, R = 3, static pushover.

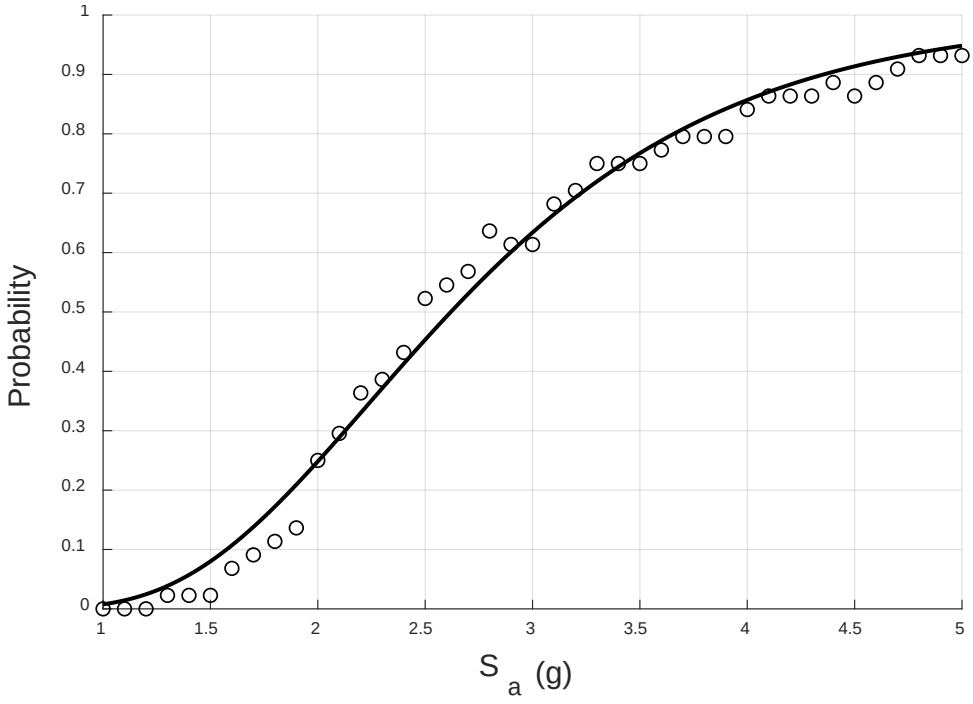


Figure I74. Archetype 73 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

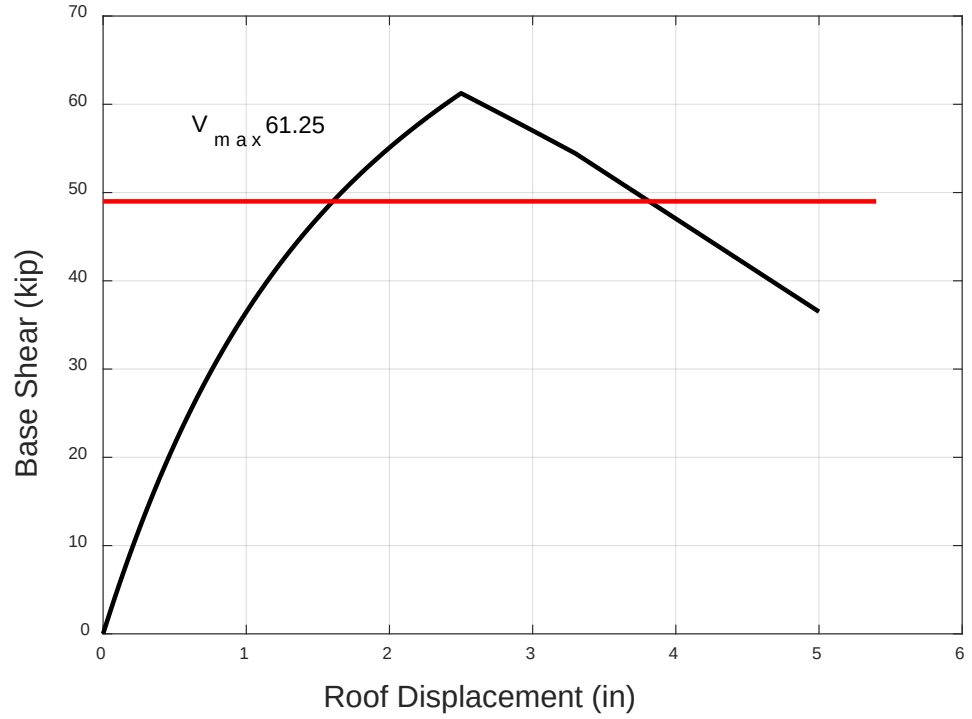


Figure I75. Archetype 74, R = 3, static pushover.

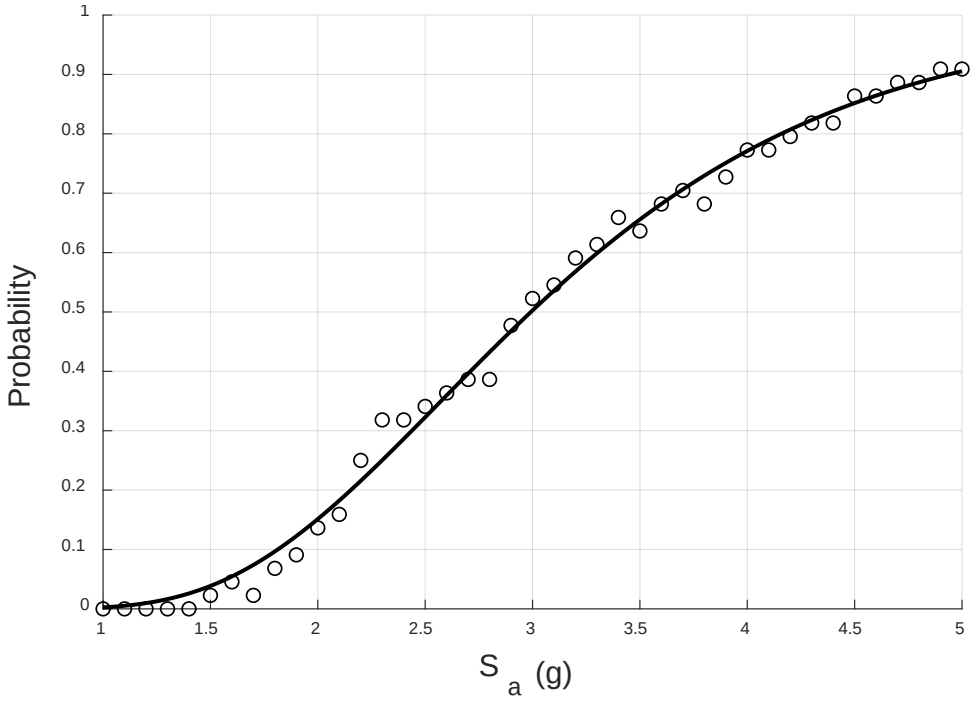


Figure I76. Archetype 74 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

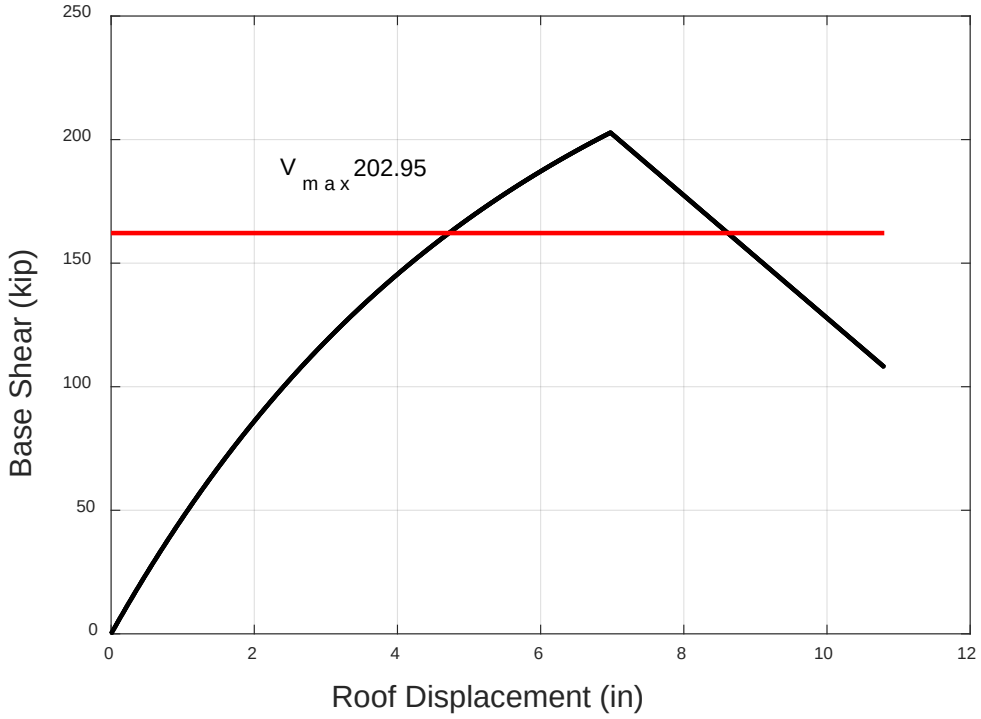


Figure I77. Archetype 75, R = 3, static pushover.

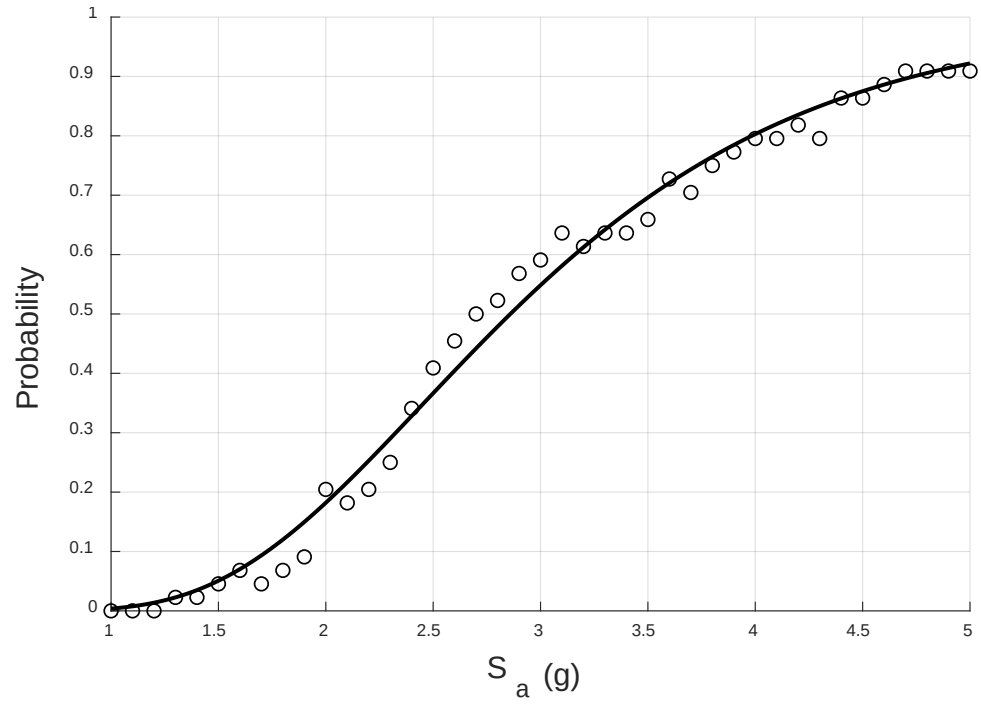


Figure I78. Archetype 75 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

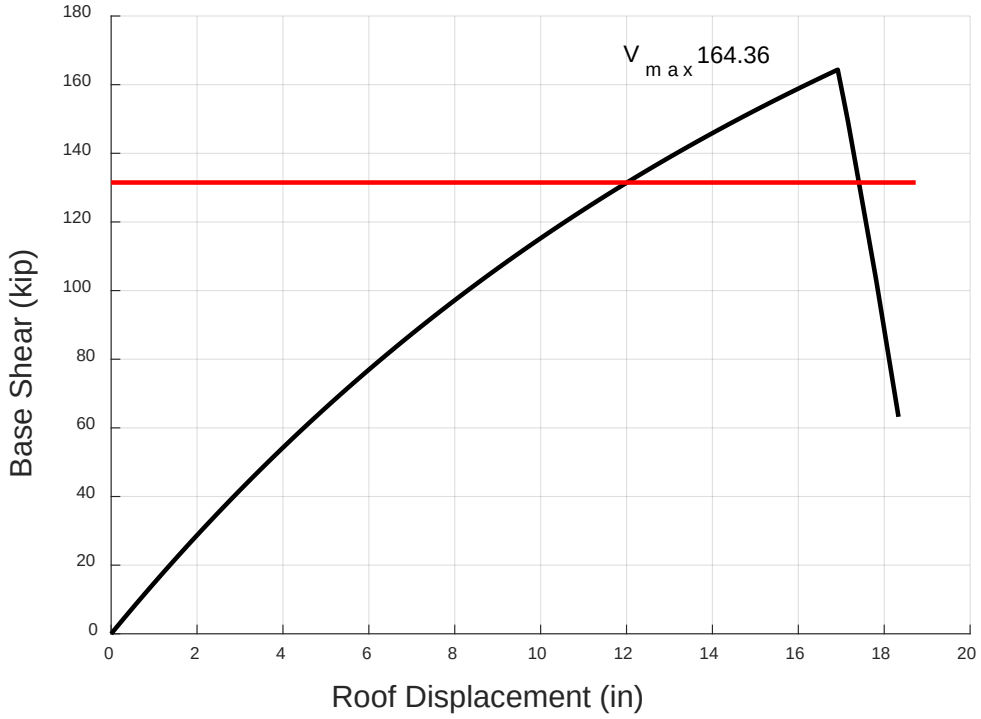


Figure I79. Archetype 76, R = 3, static pushover.

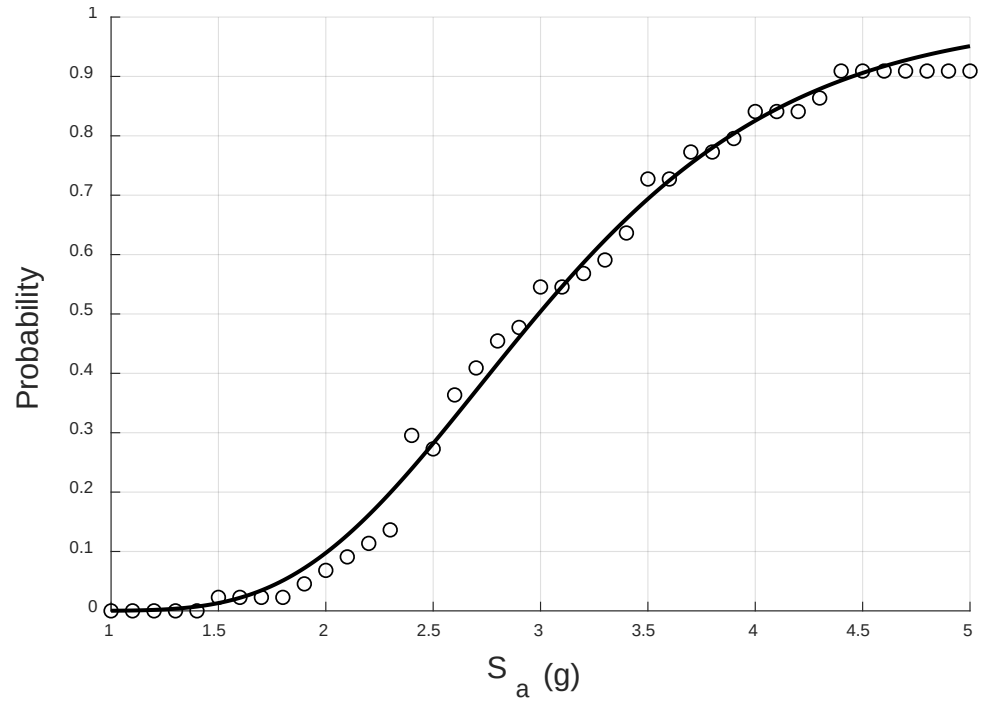


Figure I80. Archetype 76 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

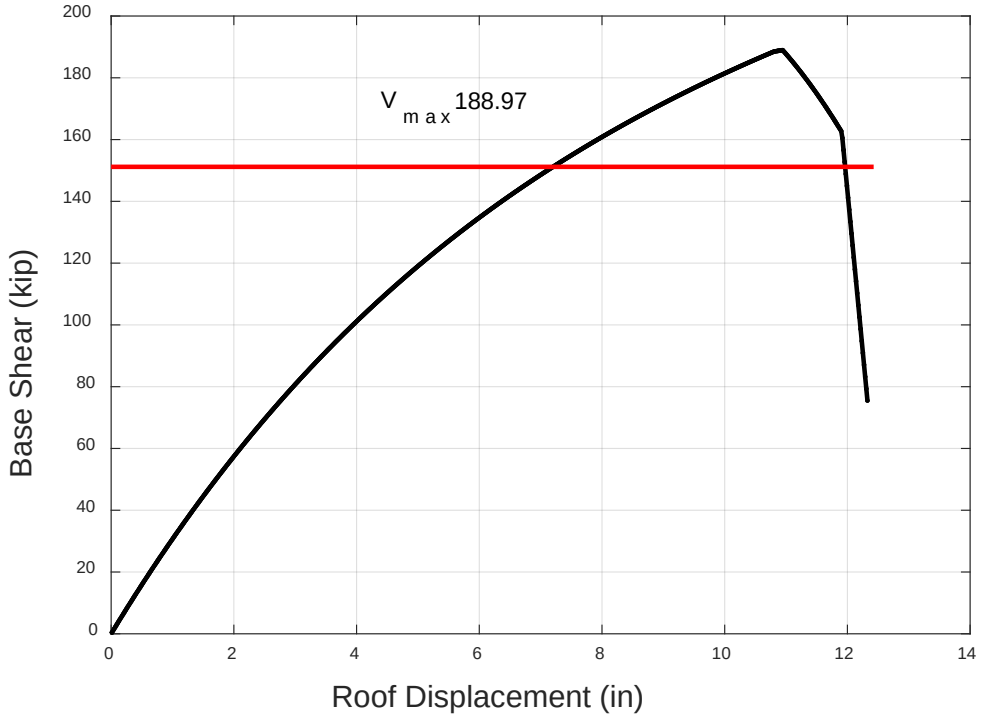


Figure I81. Archetype 77, R = 3, static pushover.

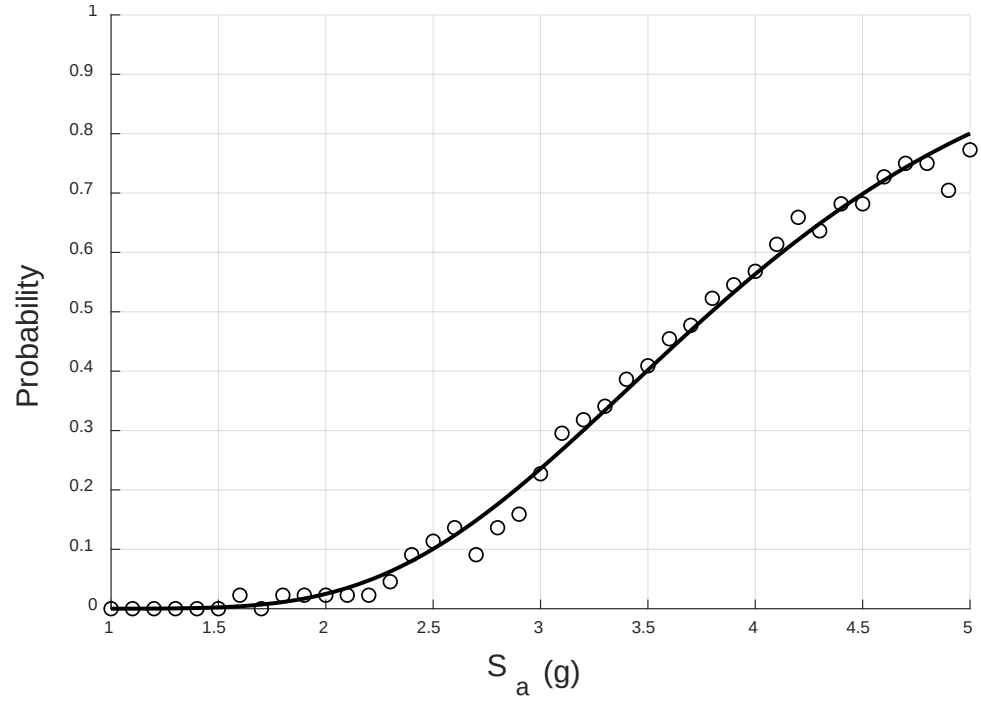


Figure I82. Archetype 77 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

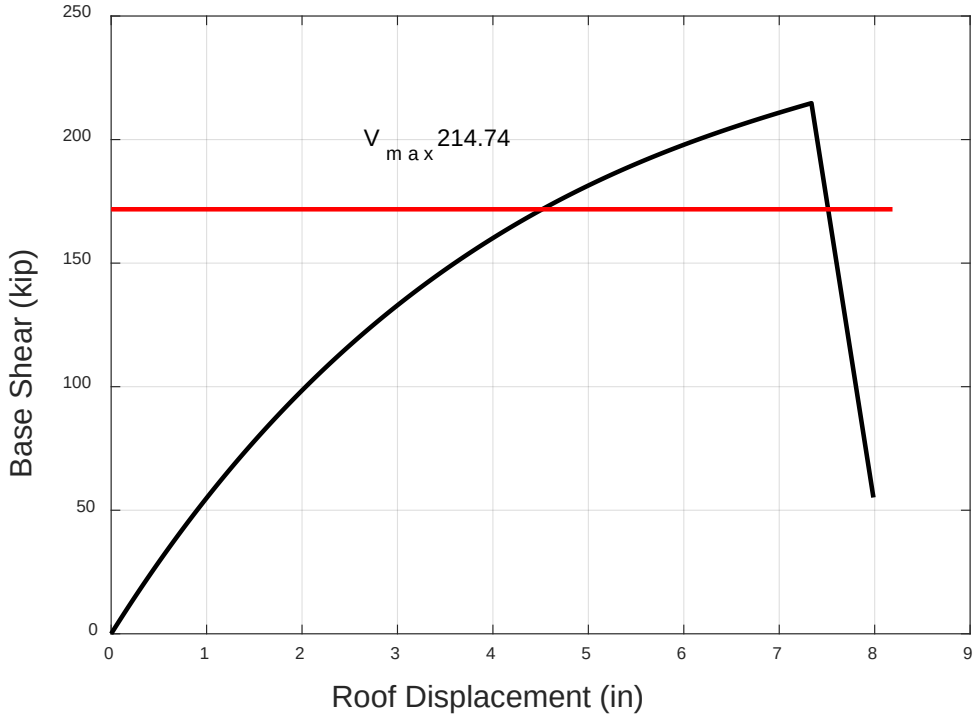


Figure I83. Archetype 78, R = 3, static pushover.

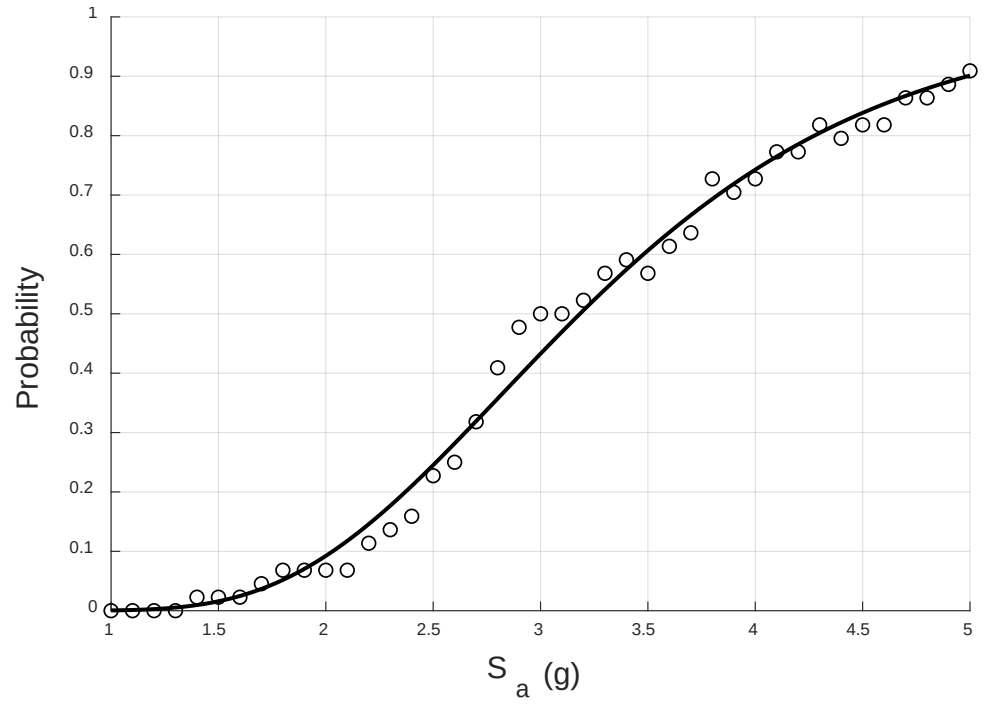


Figure I84. Archetype 78 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

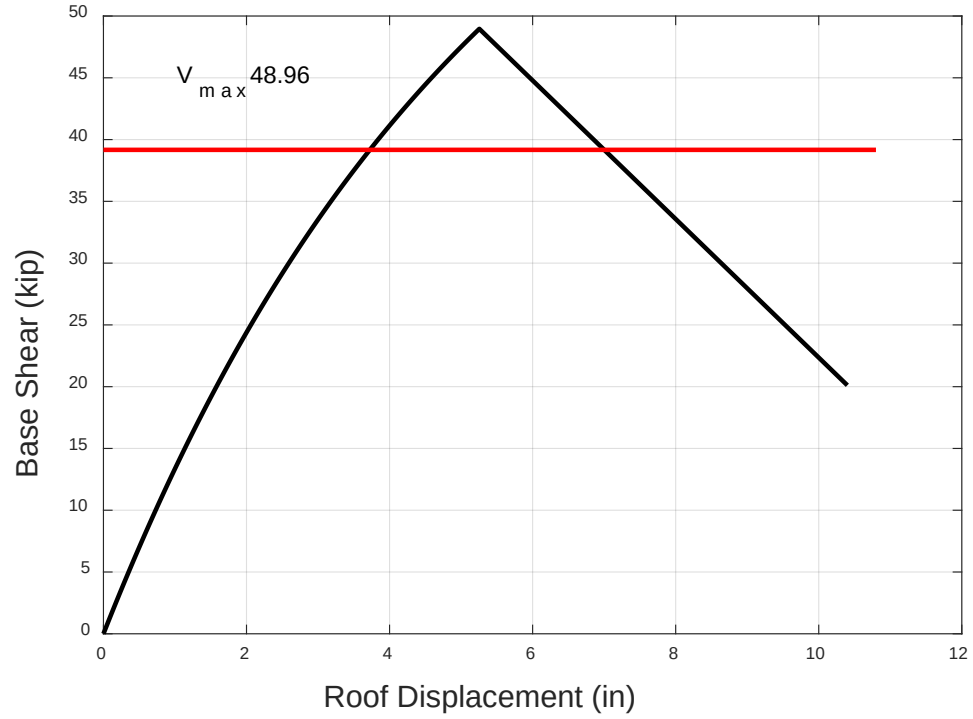


Figure I85. Archetype 85, R = 3, static pushover.

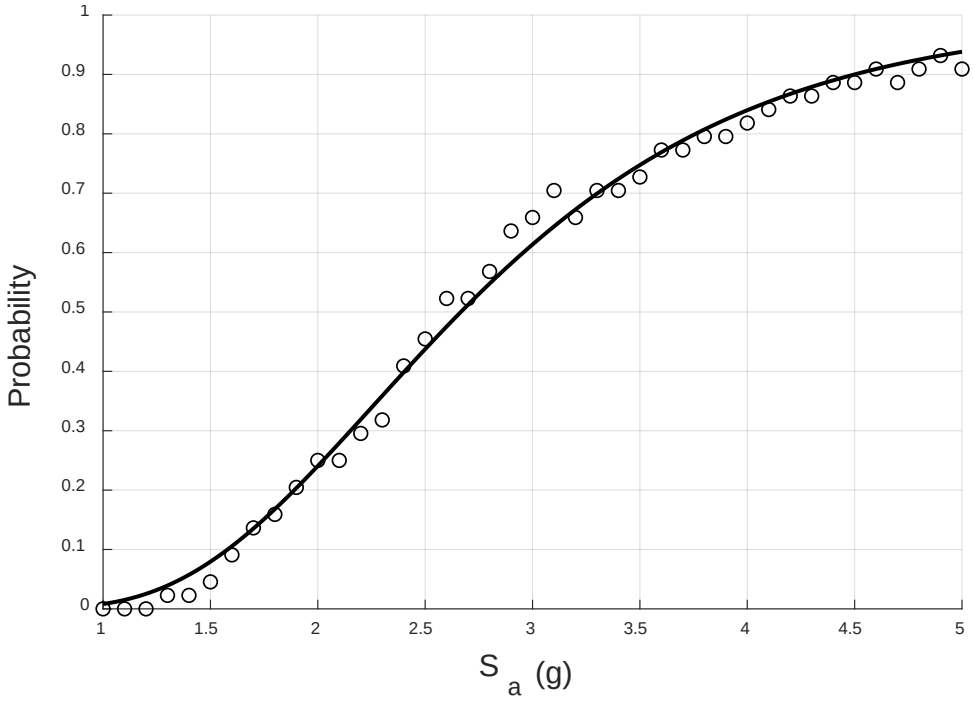


Figure I86. Archetype 85 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

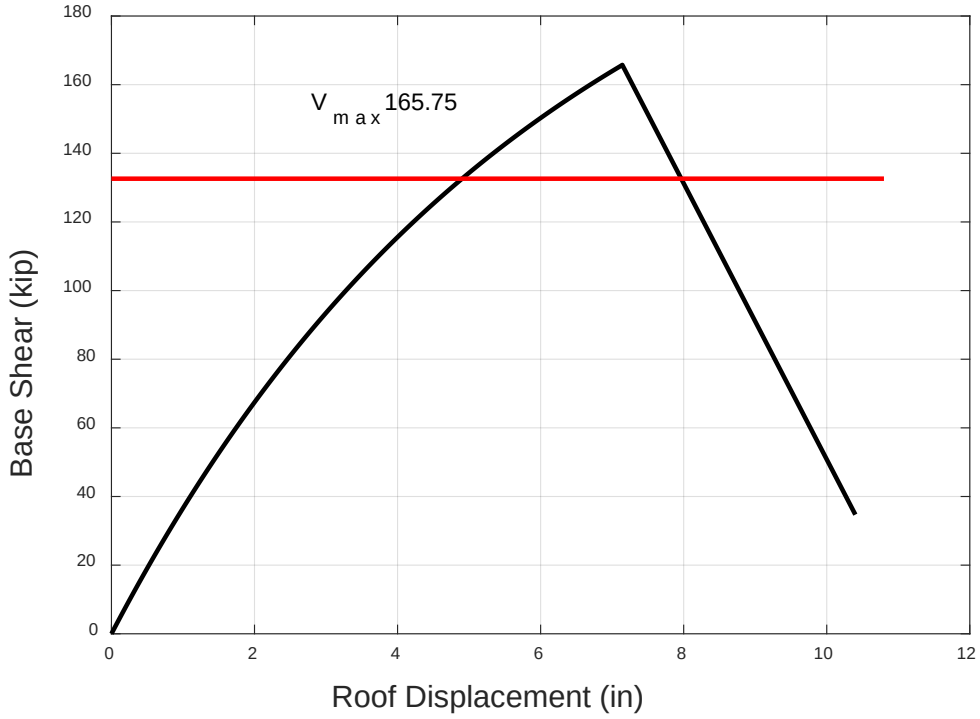


Figure I87. Archetype 87, R = 3, static pushover.

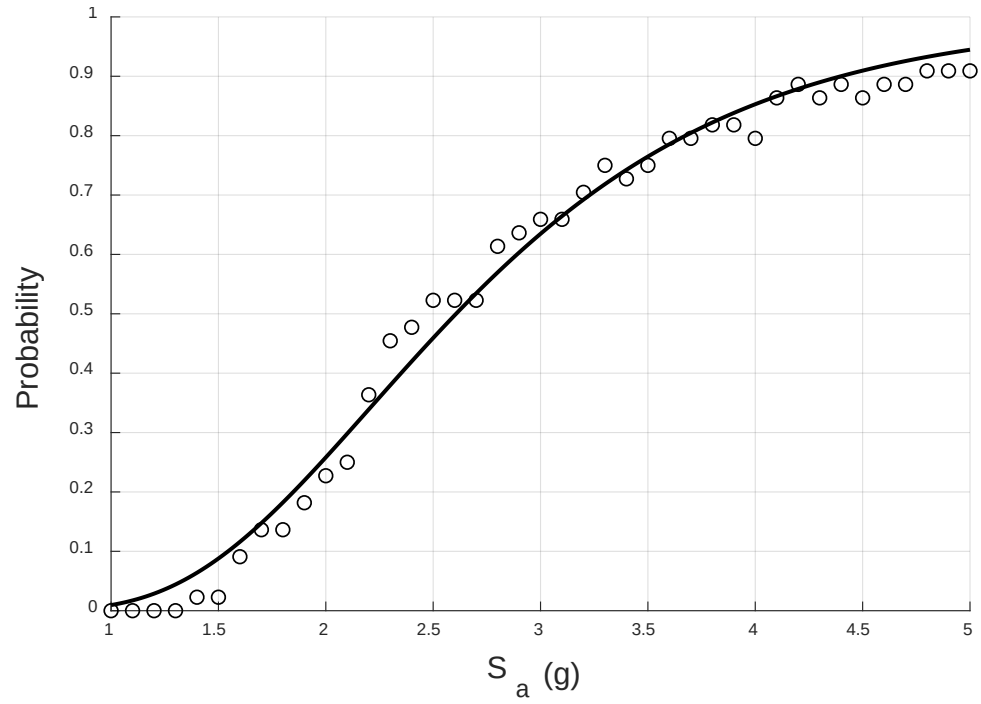


Figure I88. Archetype 87 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

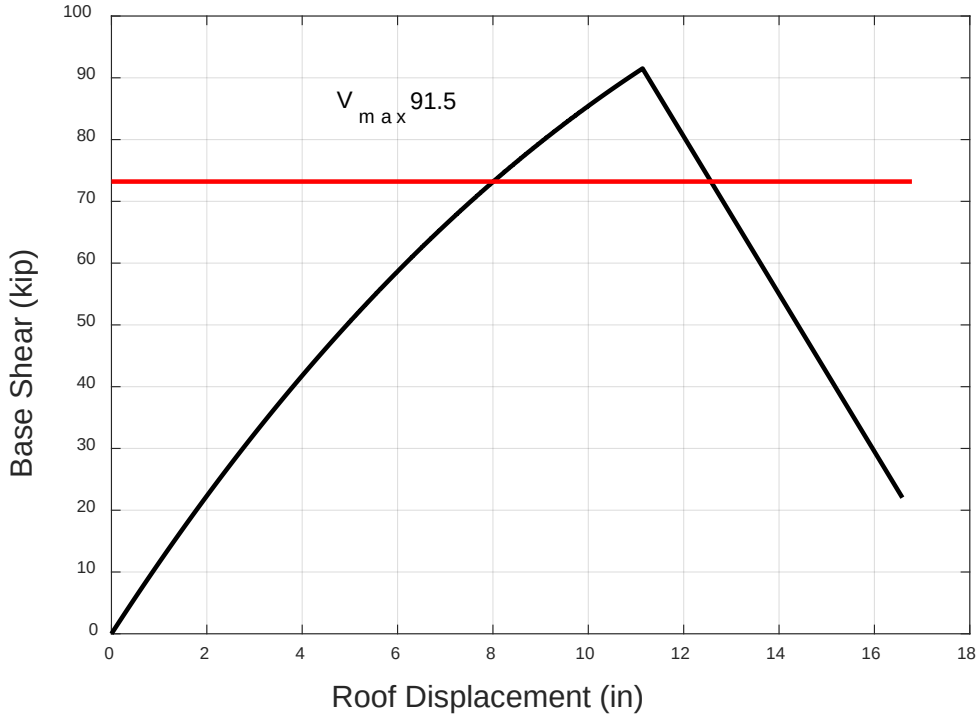


Figure I89. Archetype 88, R = 3, static pushover.

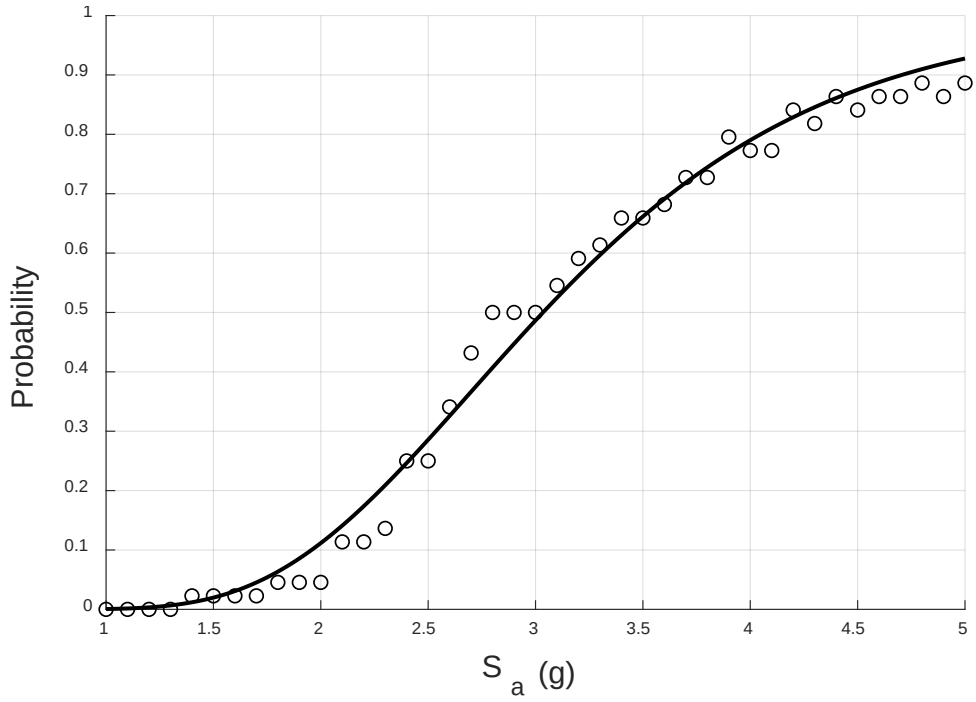


Figure I90. Archetype 88 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

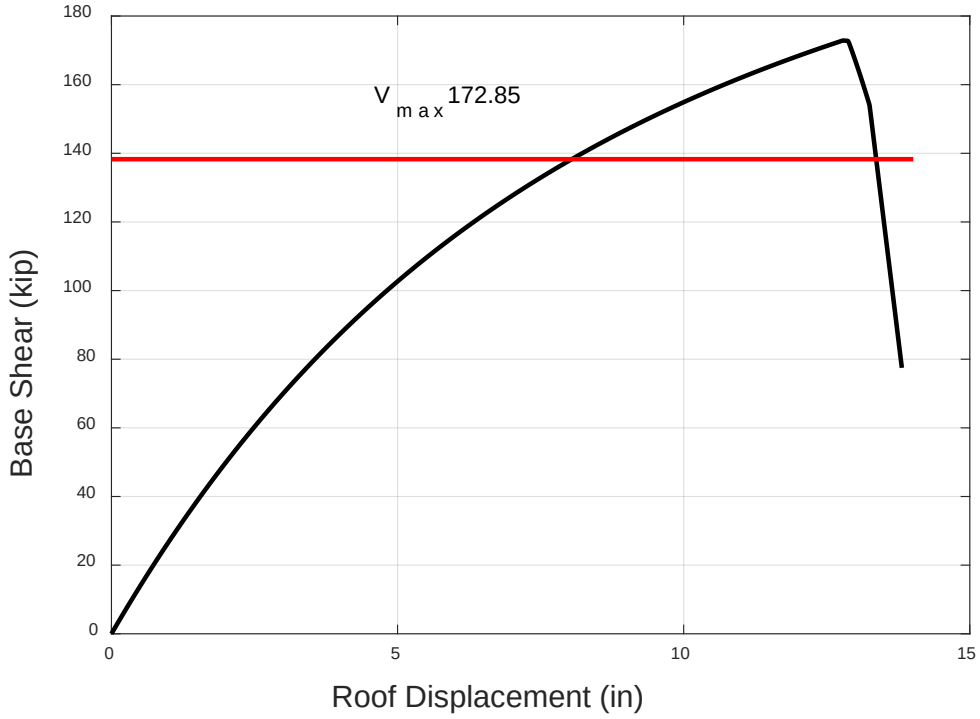


Figure I91. Archetype 89, R = 3, static pushover.

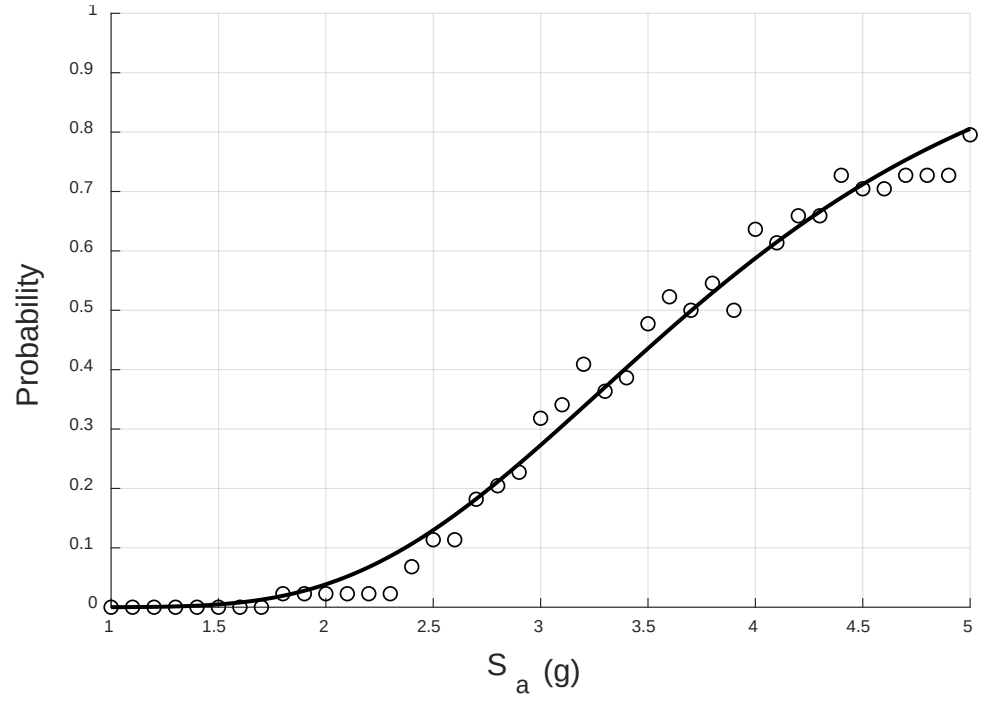


Figure I92. Archetype 89 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

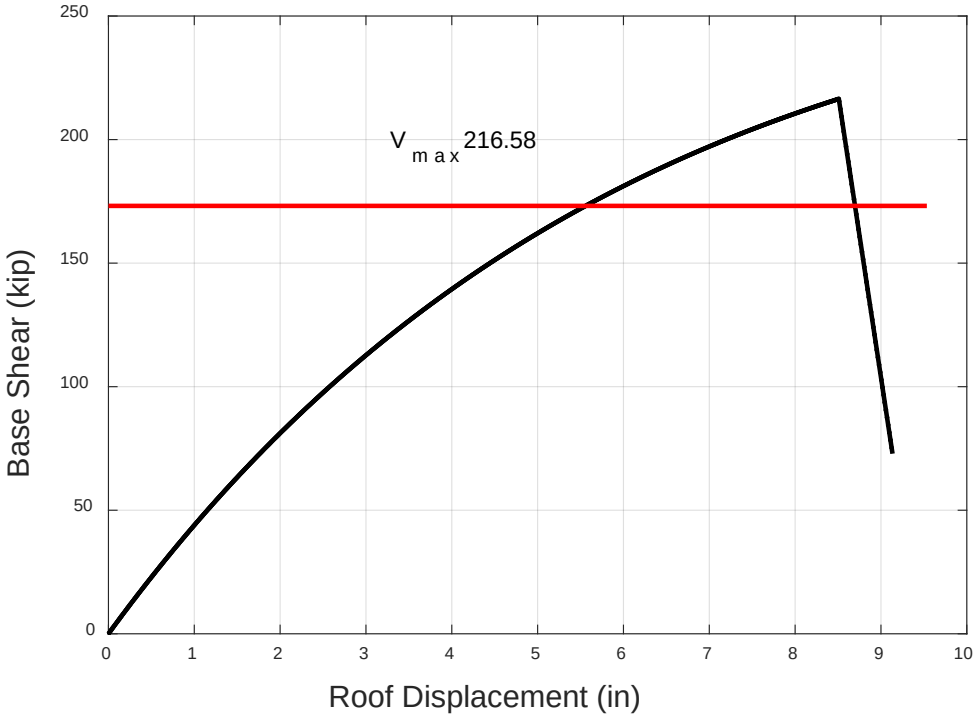


Figure I93. Archetype 90, R = 3, static pushover.

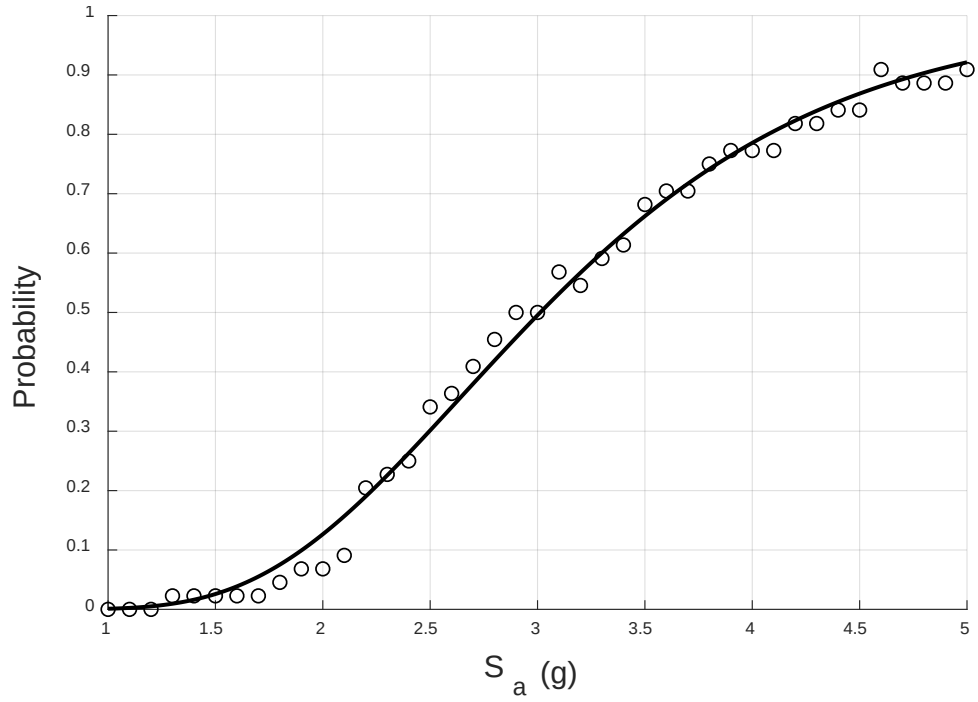


Figure I94. Archetype 90 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

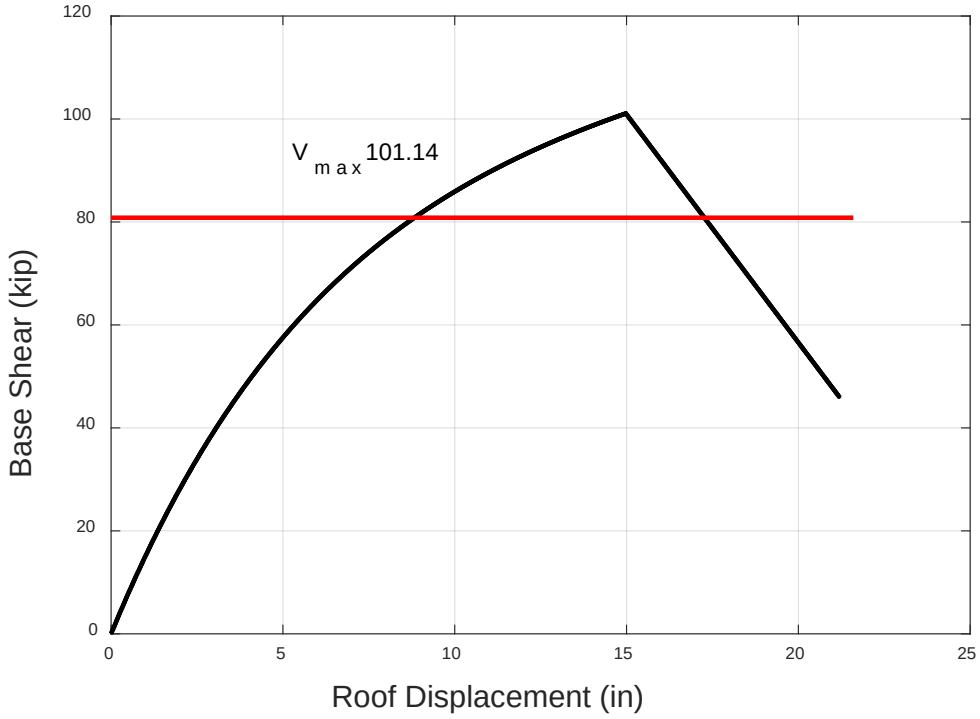


Figure I95. Archetype 97, R = 3, static pushover.

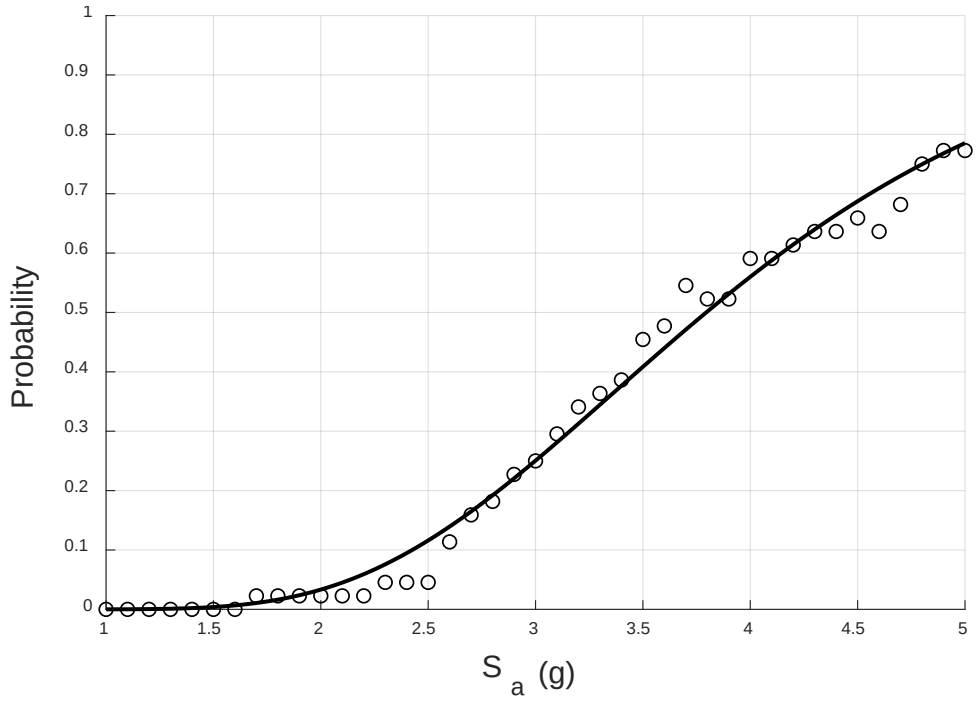


Figure I96. Archetype 97 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

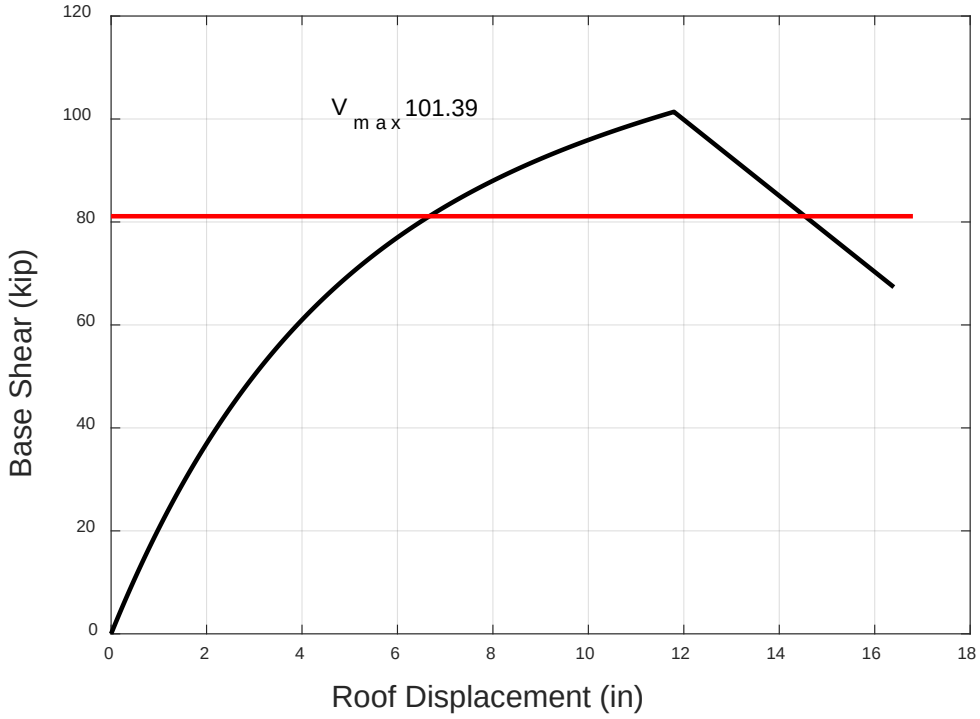


Figure I97. Archetype 98, R = 3, static pushover.

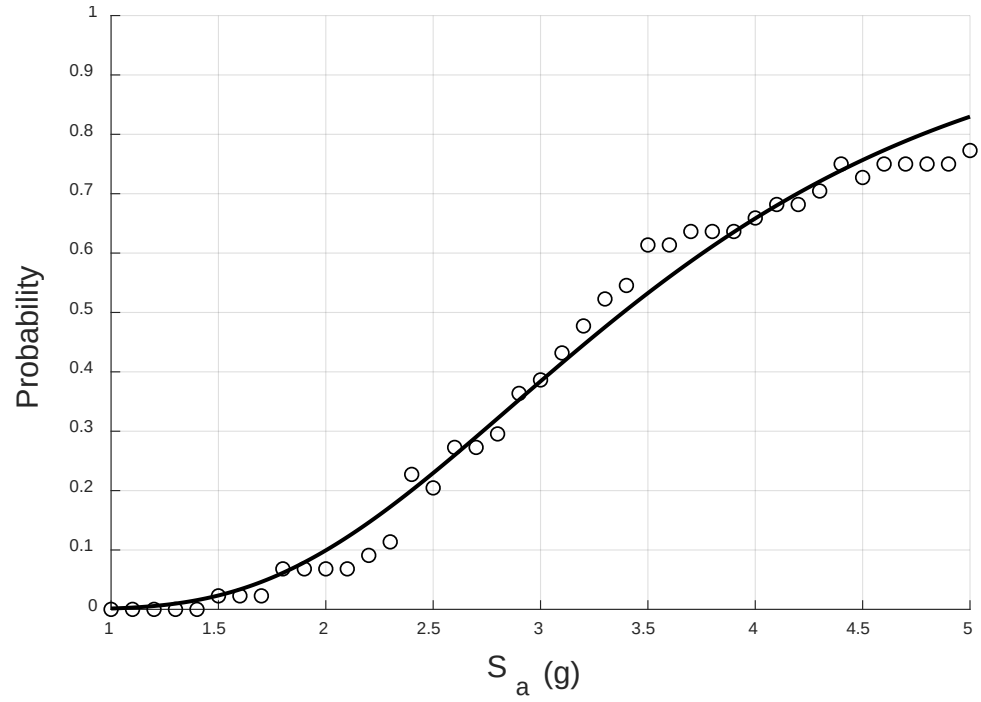


Figure I98. Archetype 98 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

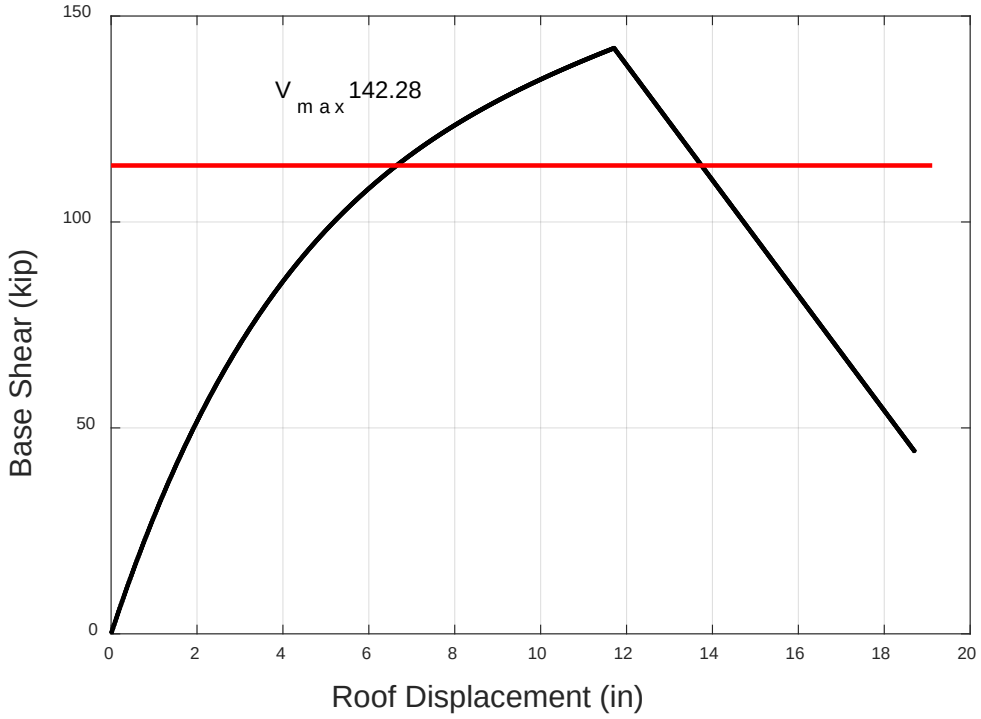


Figure I99. Archetype 99, R = 3, static pushover.

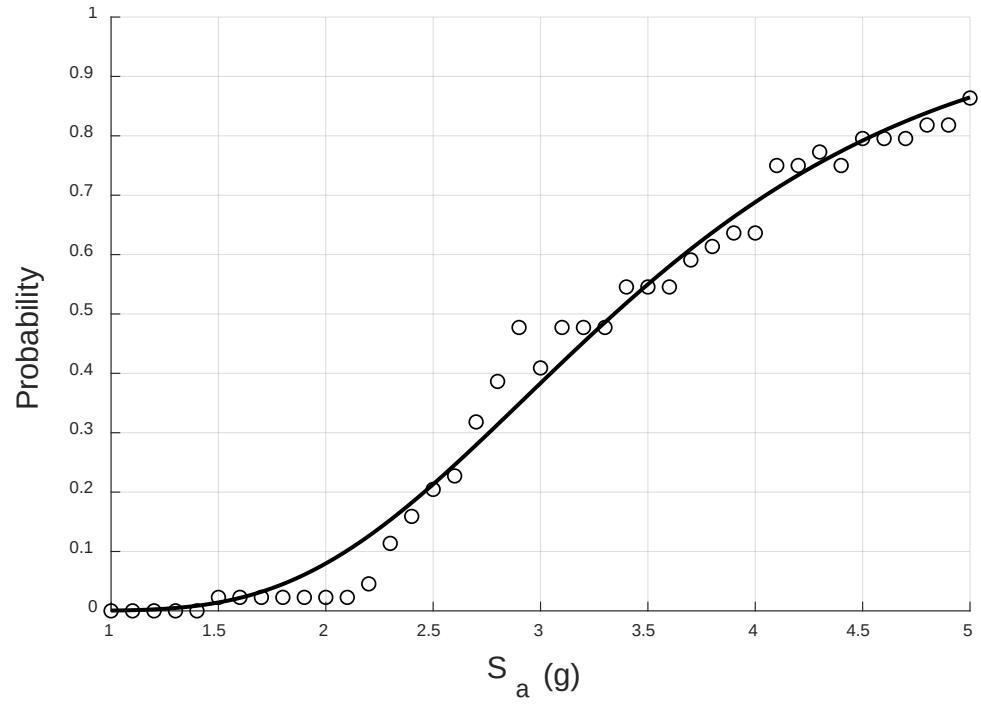


Figure I100. Archetype 99 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

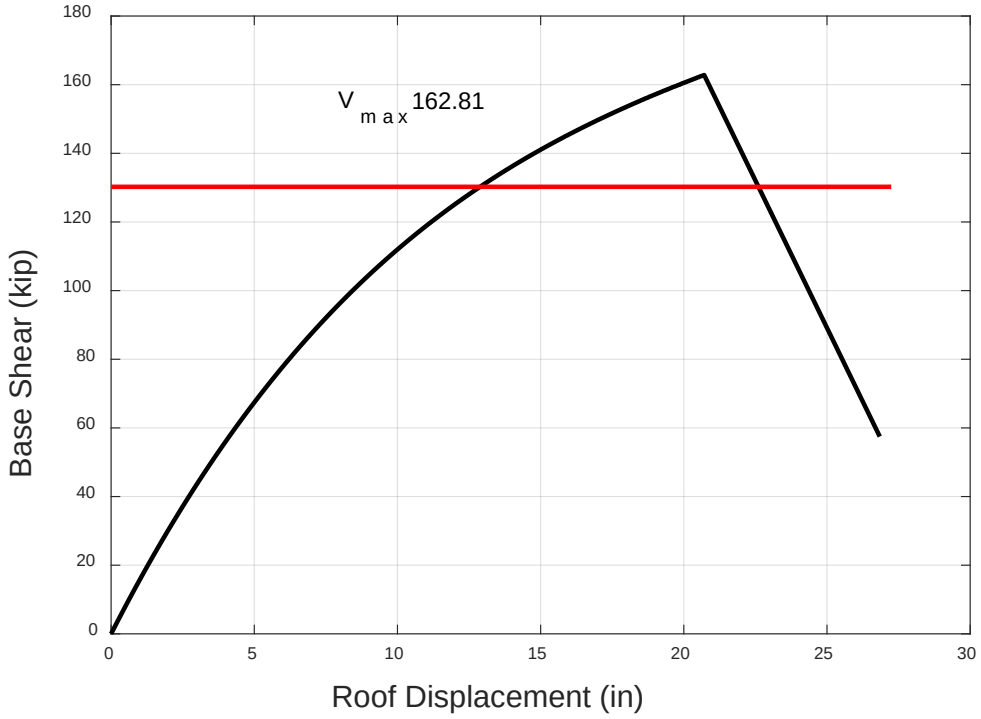


Figure I101. Archetype 100, R = 3, static pushover.

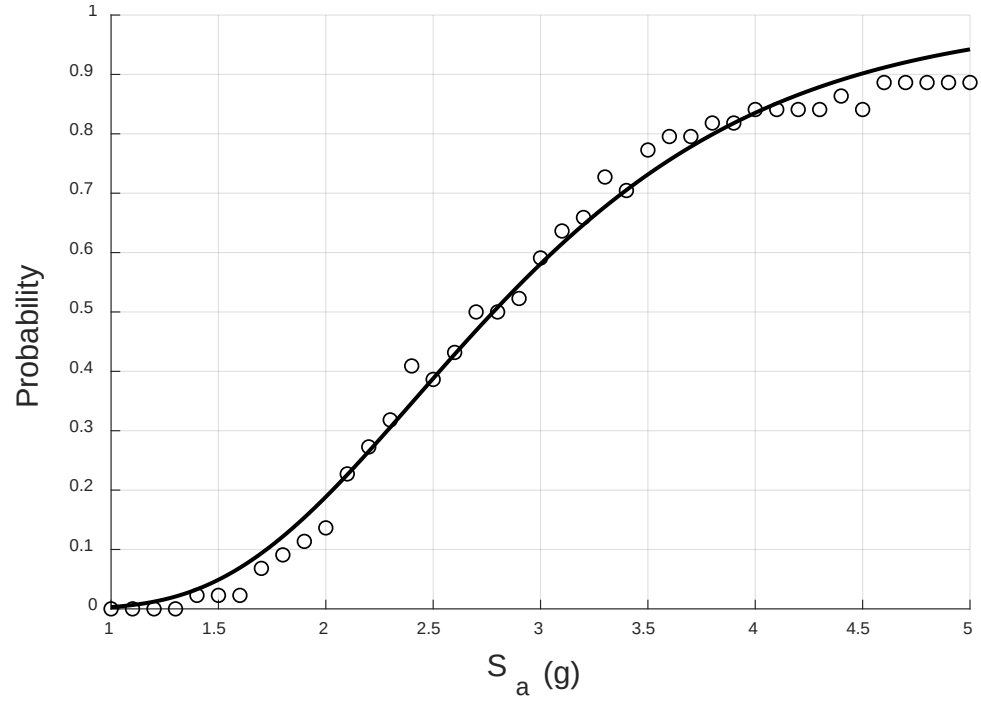


Figure I102. Archetype 100 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

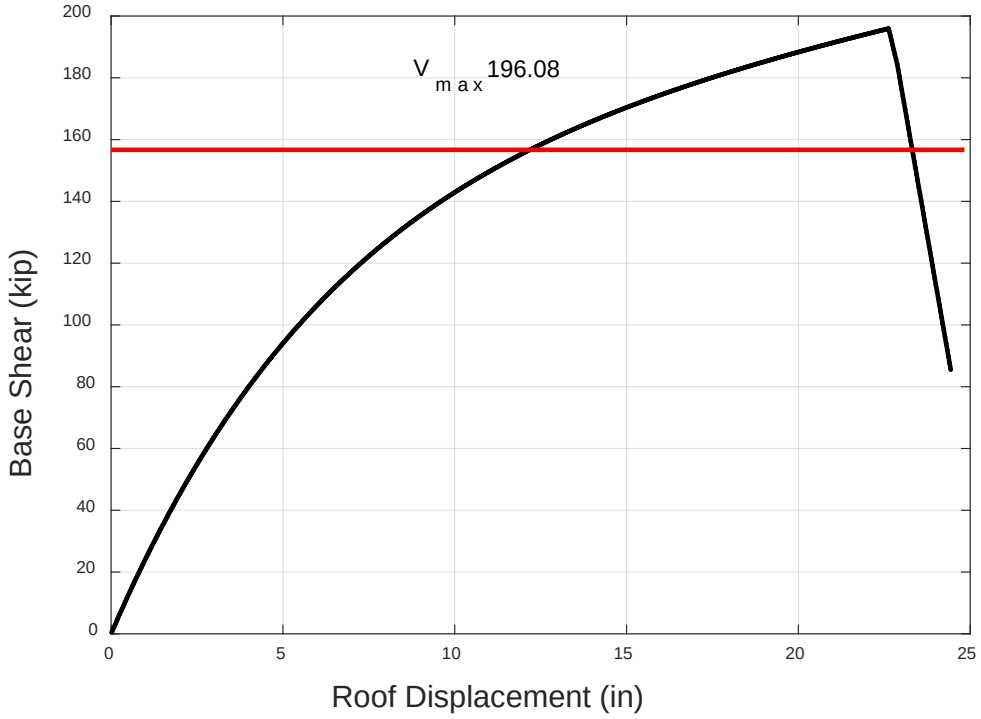


Figure I103. Archetype 101, R = 3, static pushover.

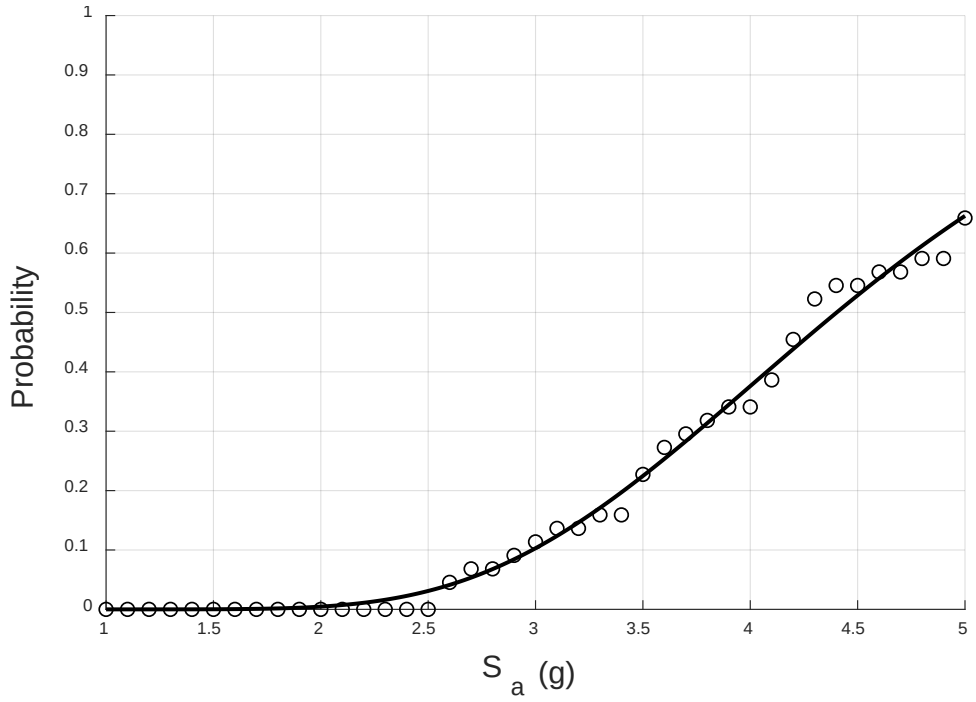


Figure I104. Archetype 101 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

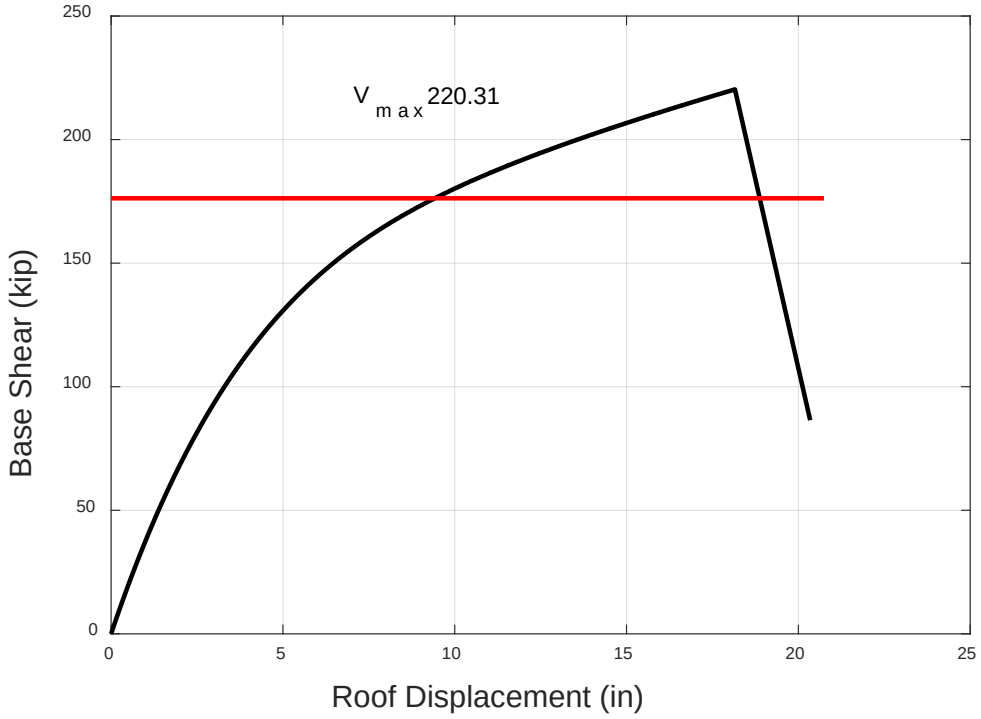


Figure I105. Archetype 102, R = 3, static pushover.

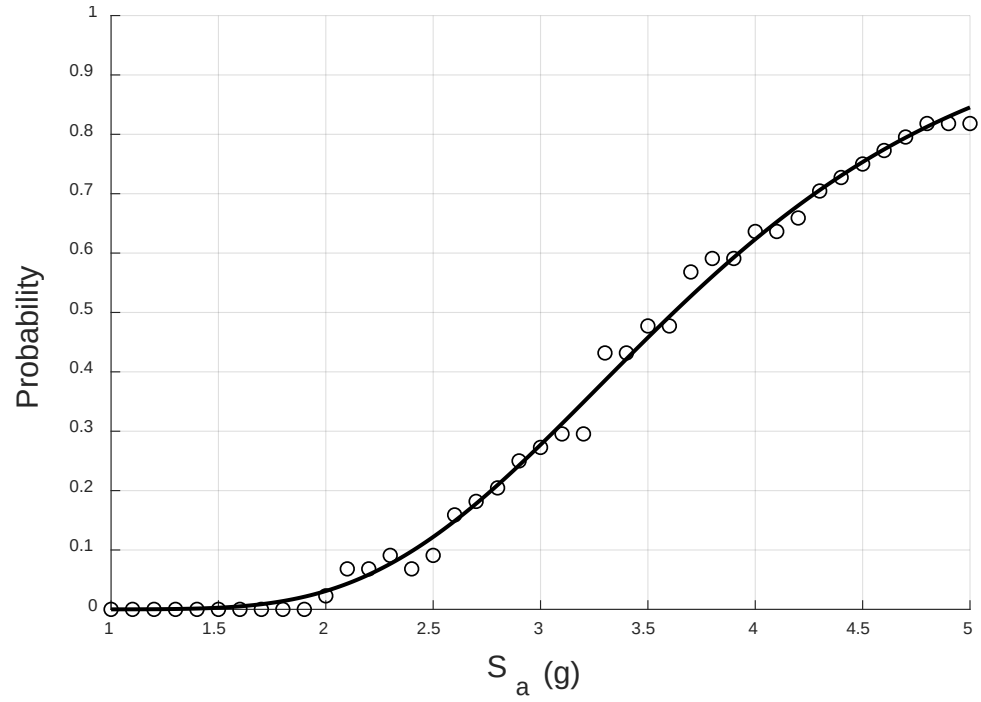


Figure I106. Archetype 102 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

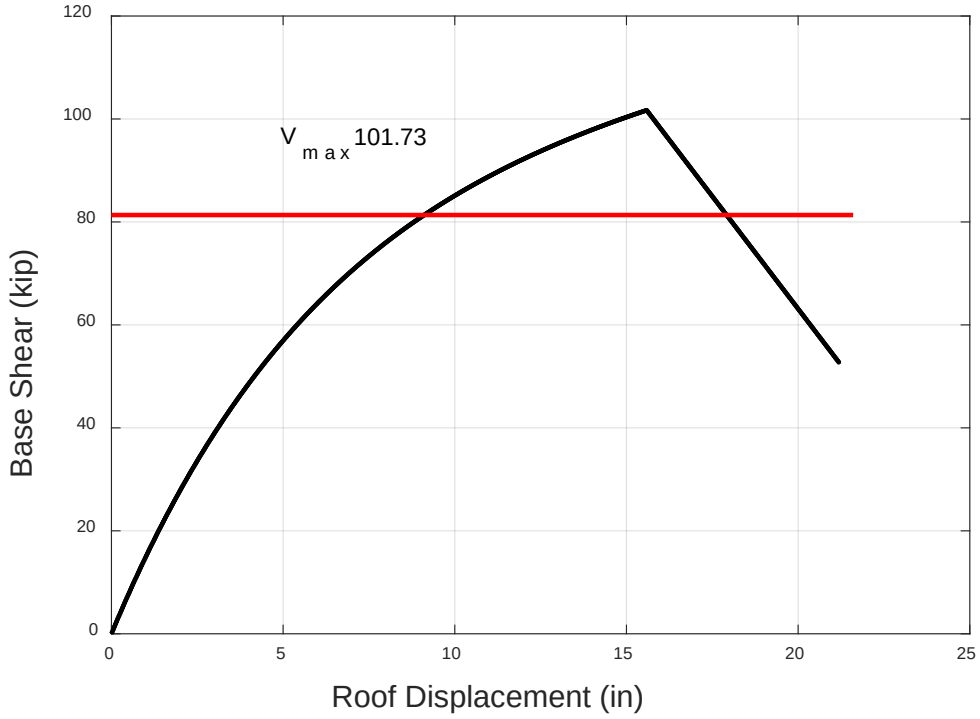


Figure I107. Archetype 109, R = 3, static pushover.

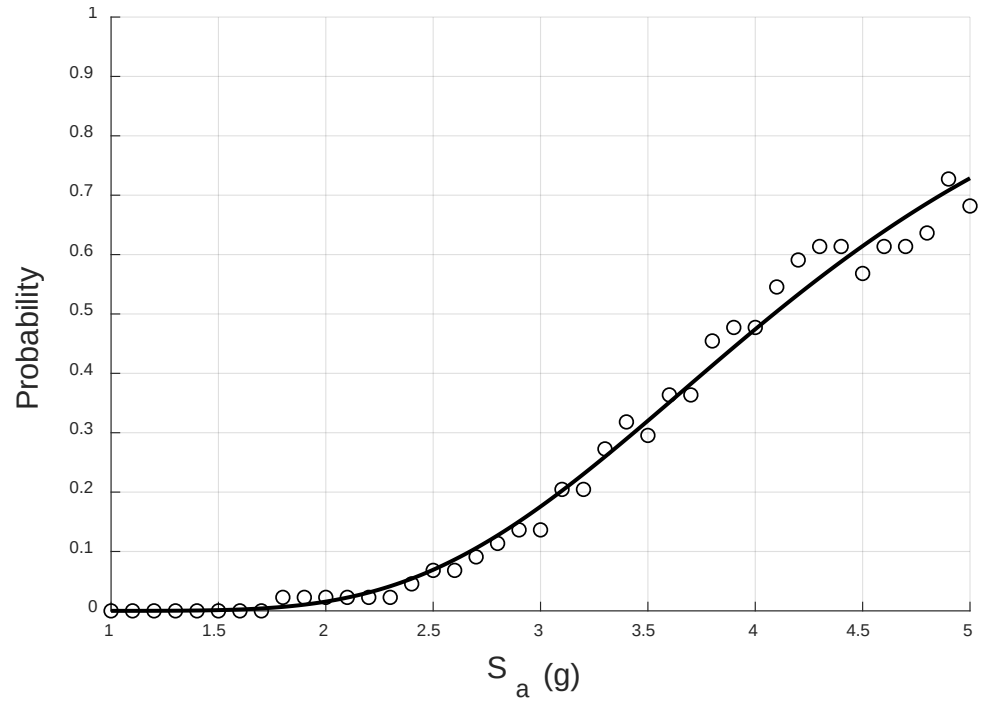


Figure I108. Archetype 109 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

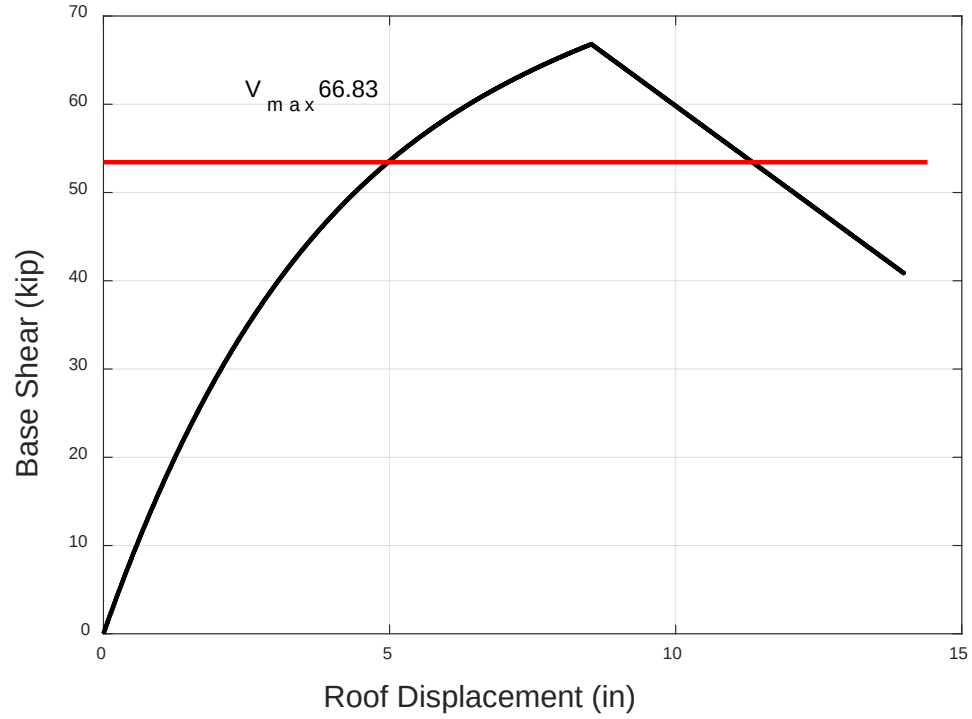


Figure I109. Archetype 110, R = 3, static pushover.

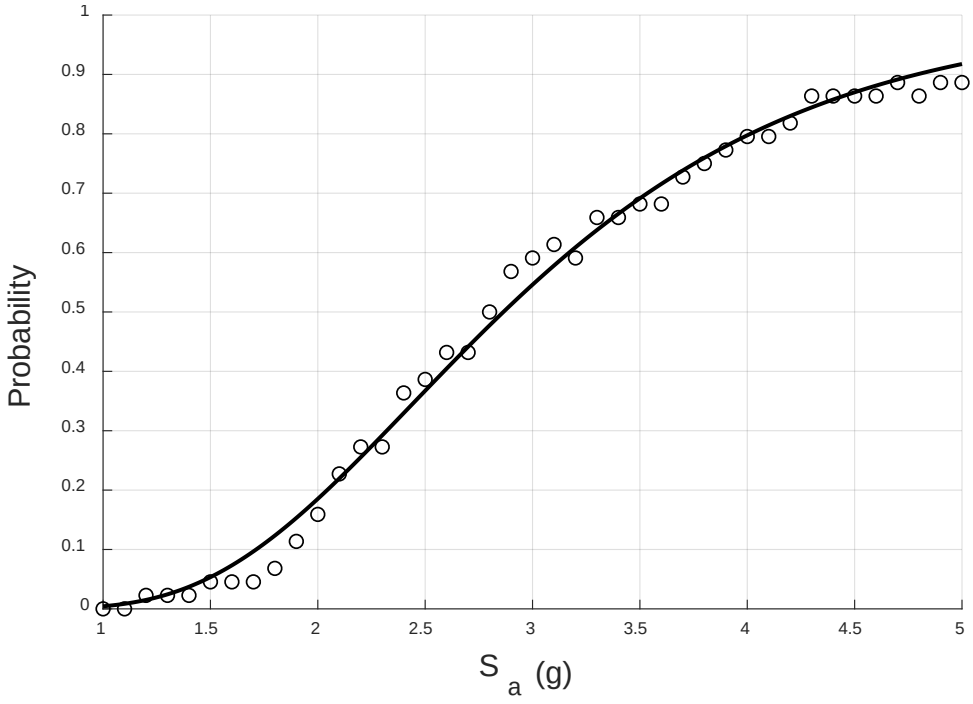


Figure I110. Archetype 110 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

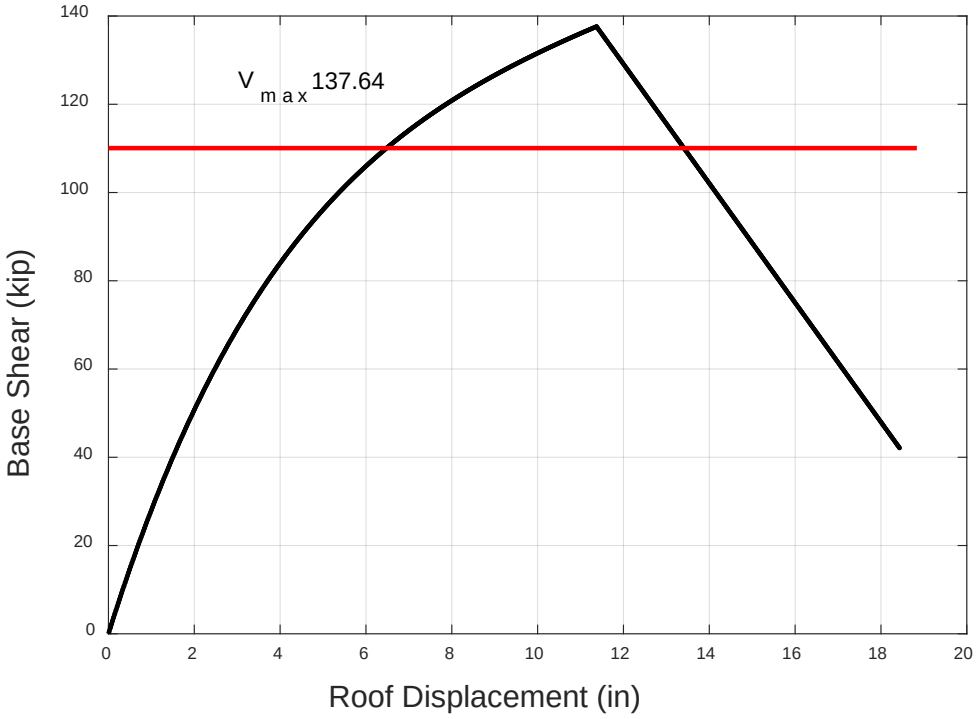


Figure I111. Archetype 111, R = 3, static pushover.

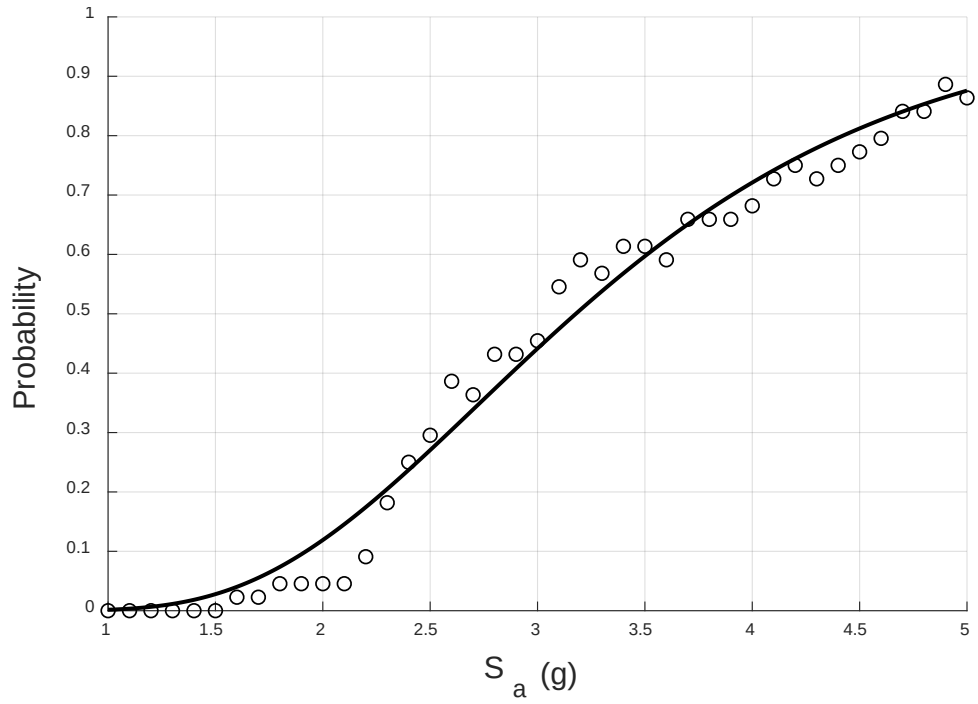


Figure I112. Archetype 111 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

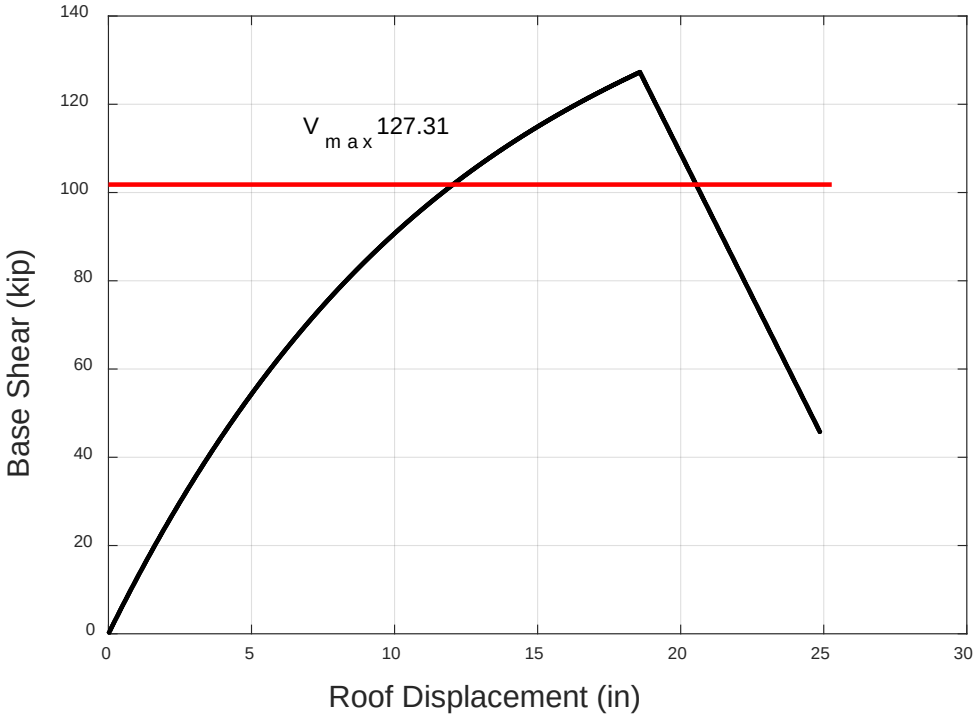


Figure I113. Archetype 112, R = 3, static pushover.

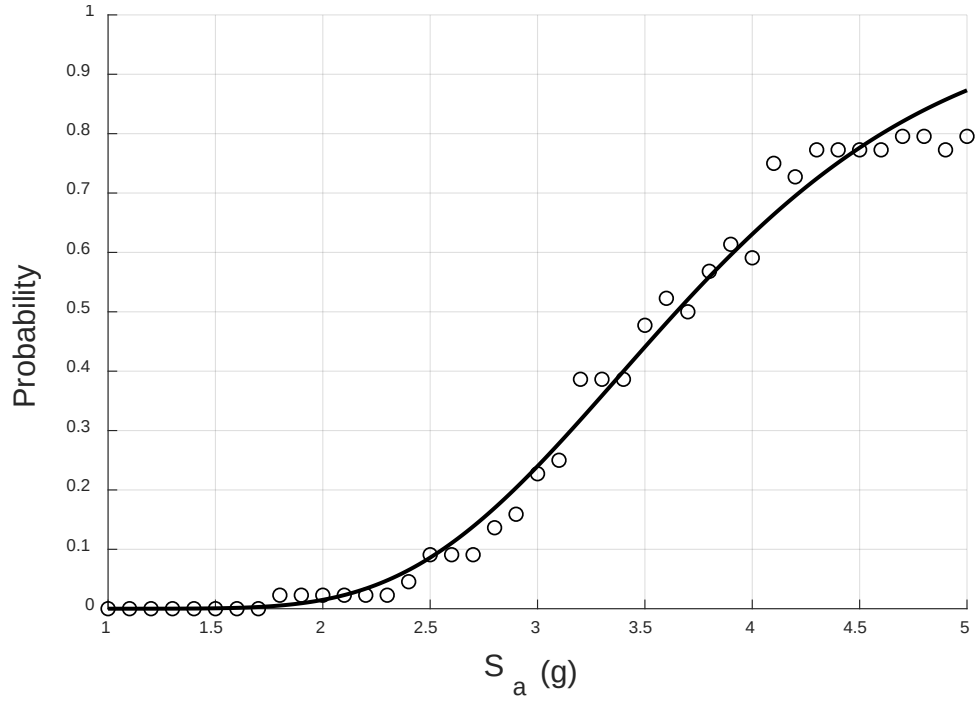


Figure I114. Archetype 112 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

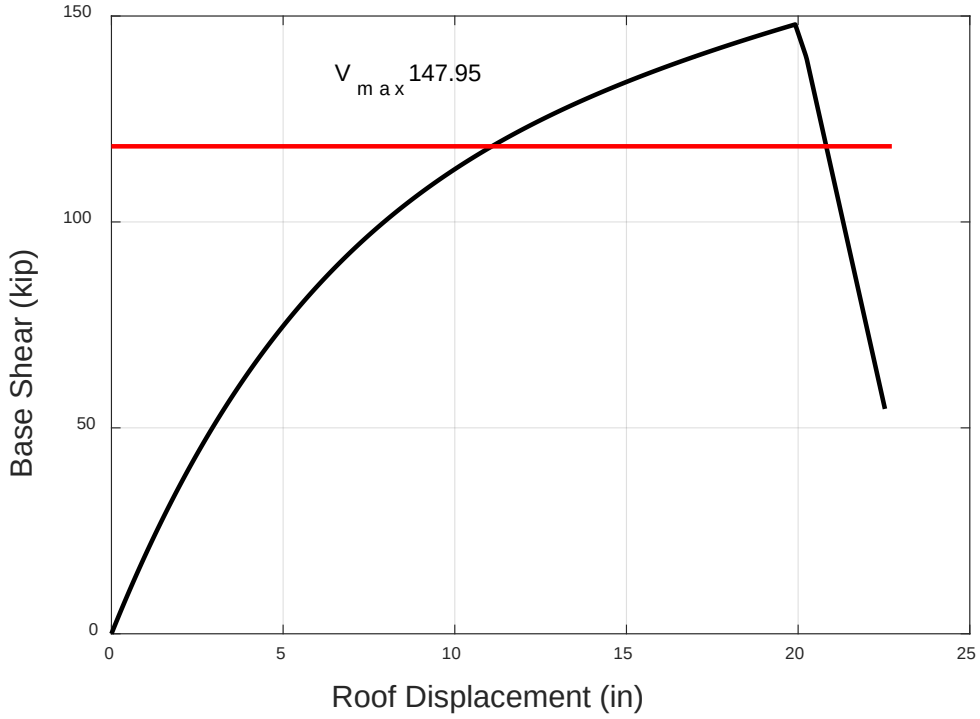


Figure I115. Archetype 113, R = 3, static pushover.

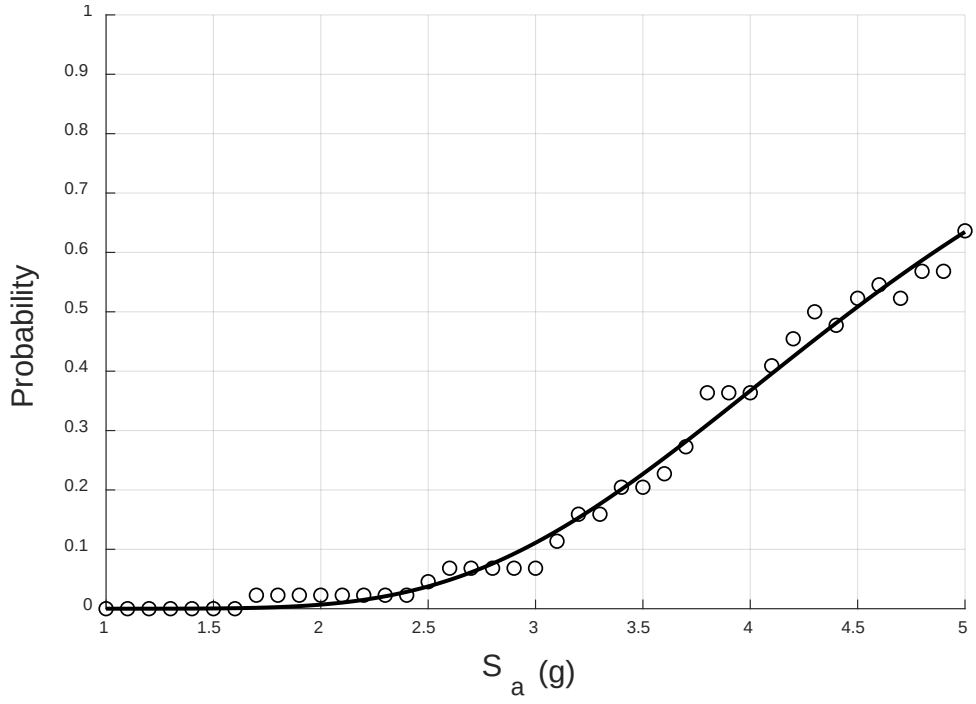


Figure I116. Archetype 113 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

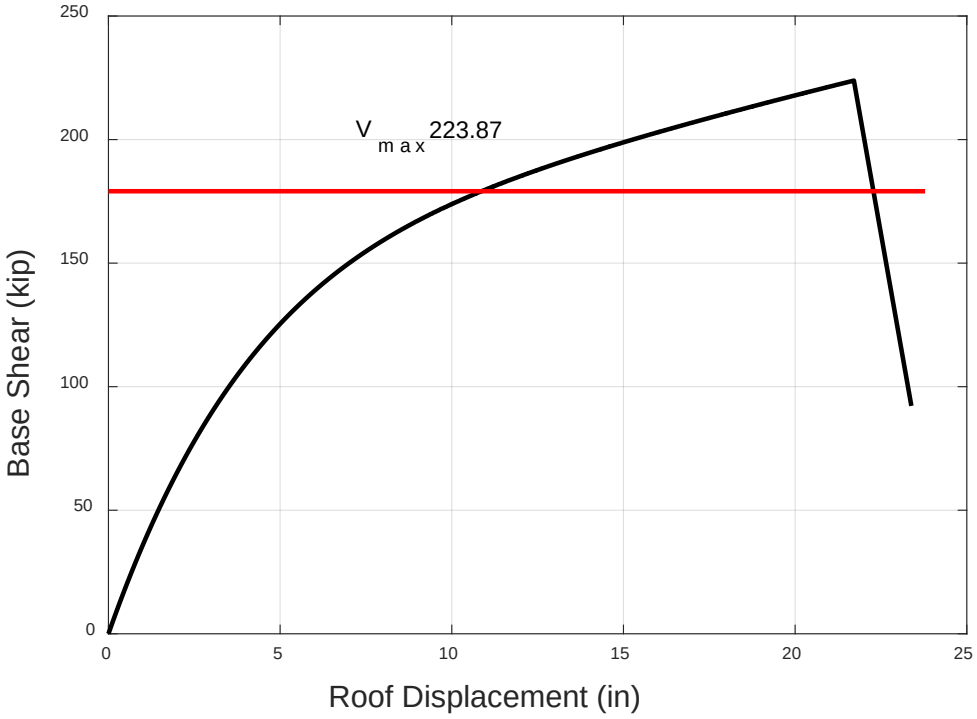


Figure I117. Archetype 114, R = 3, static pushover.

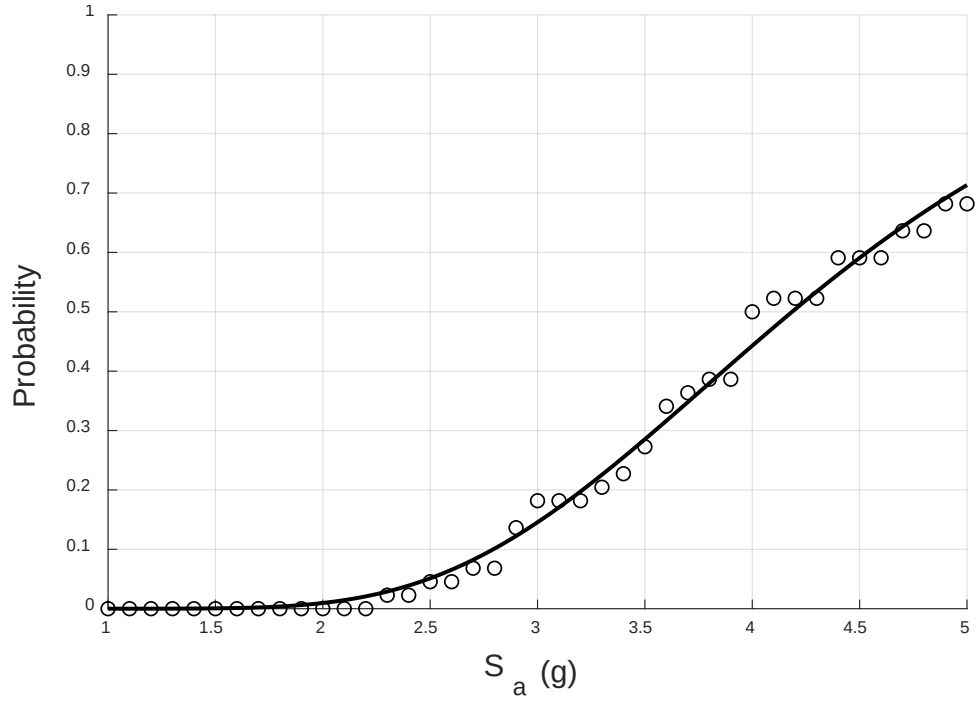


Figure I118. Archetype 114 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit.

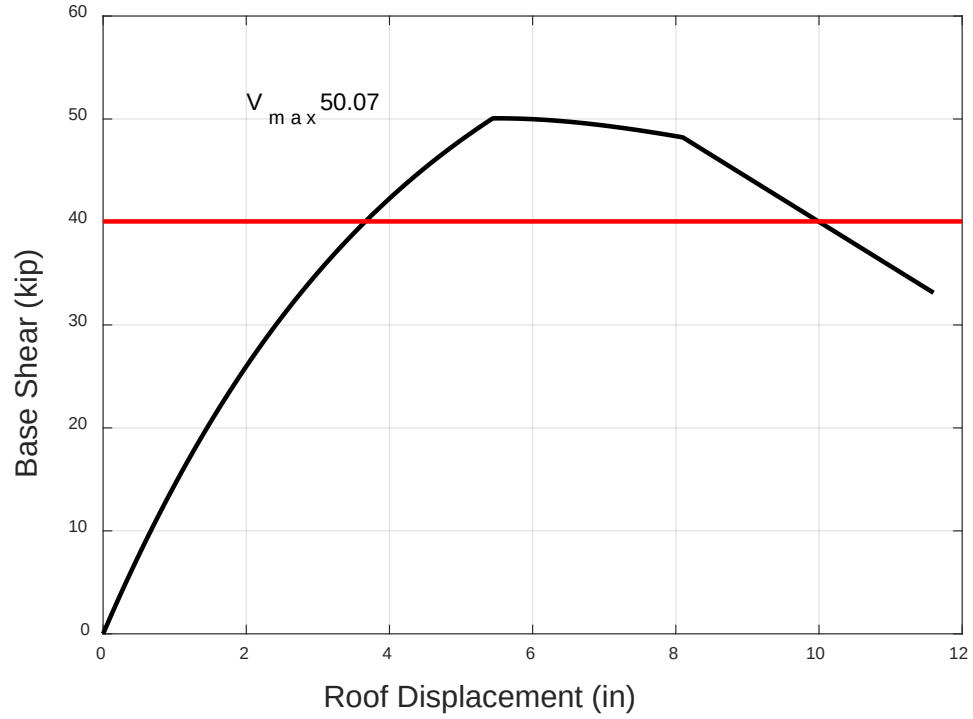


Figure I119. Archetype 121, R = 3, static pushover.

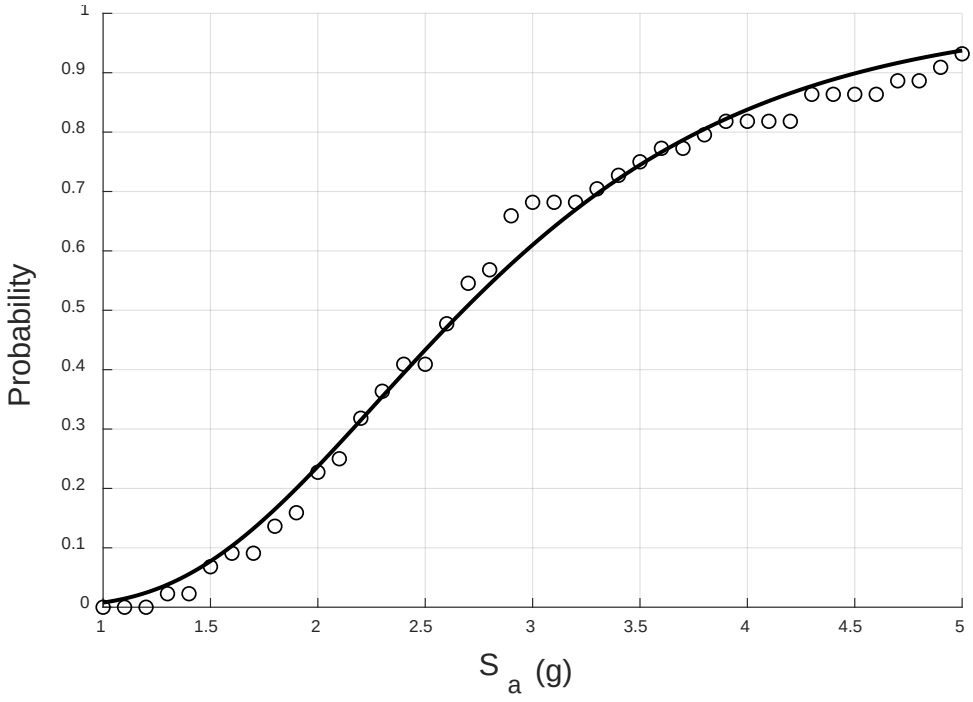


Figure I120. Archetype 121 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

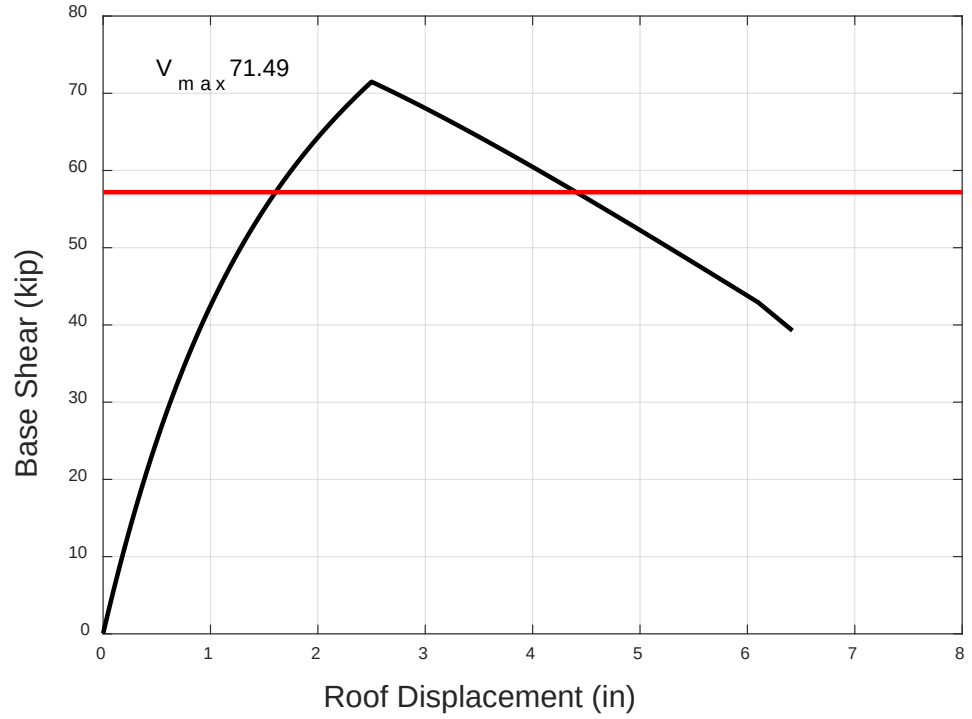


Figure I121. Archetype 122, R = 3, static pushover.

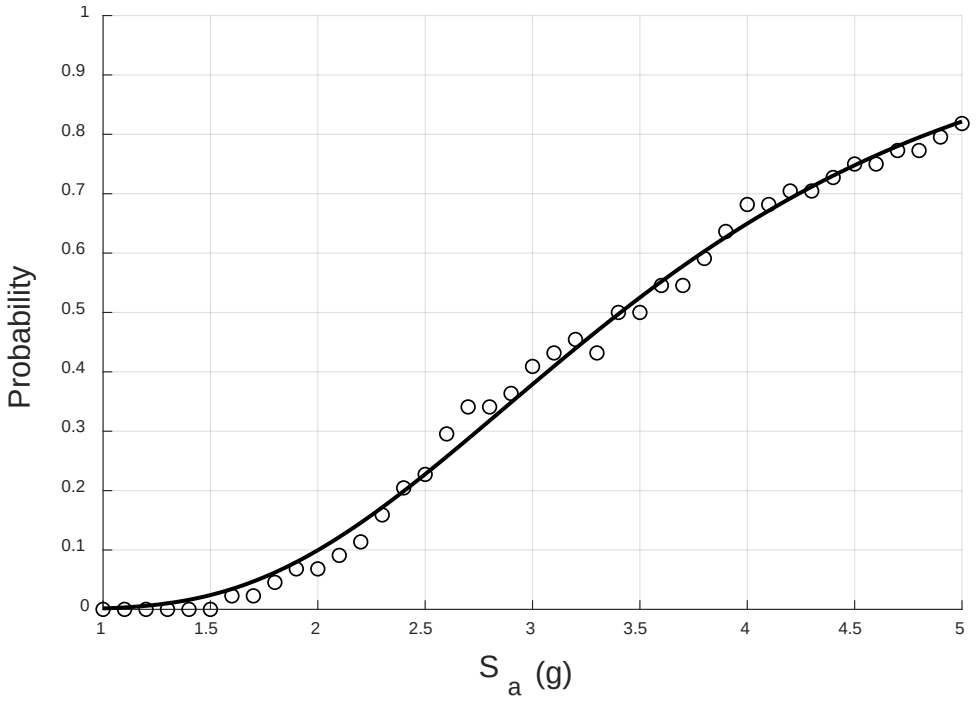


Figure I122. Archetype 122 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

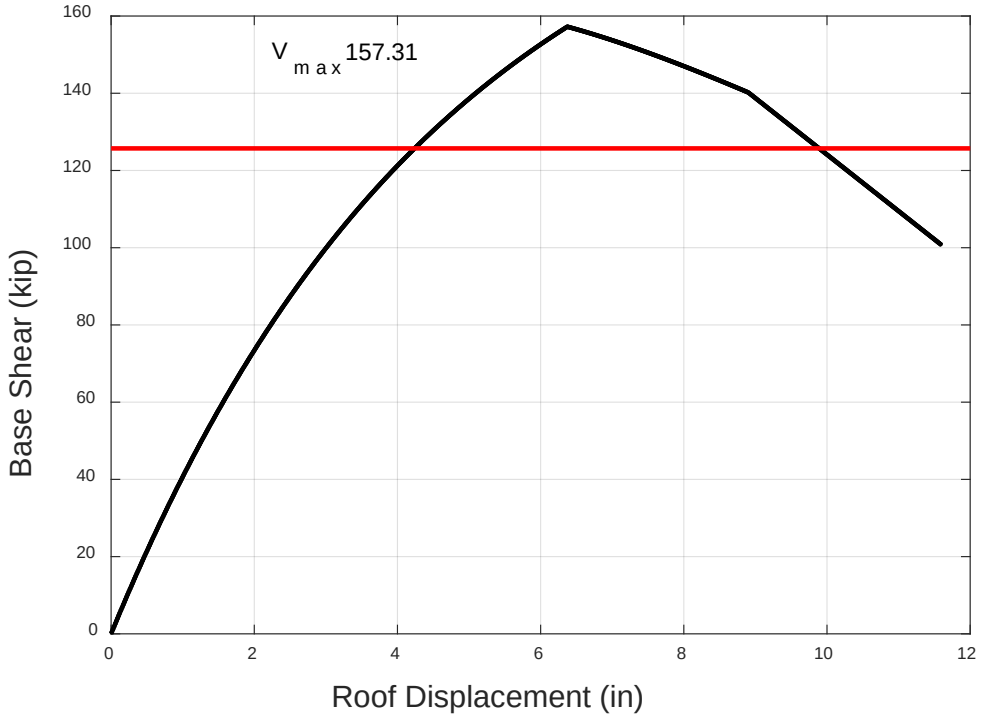


Figure I123. Archetype 123, R = 3, static pushover.

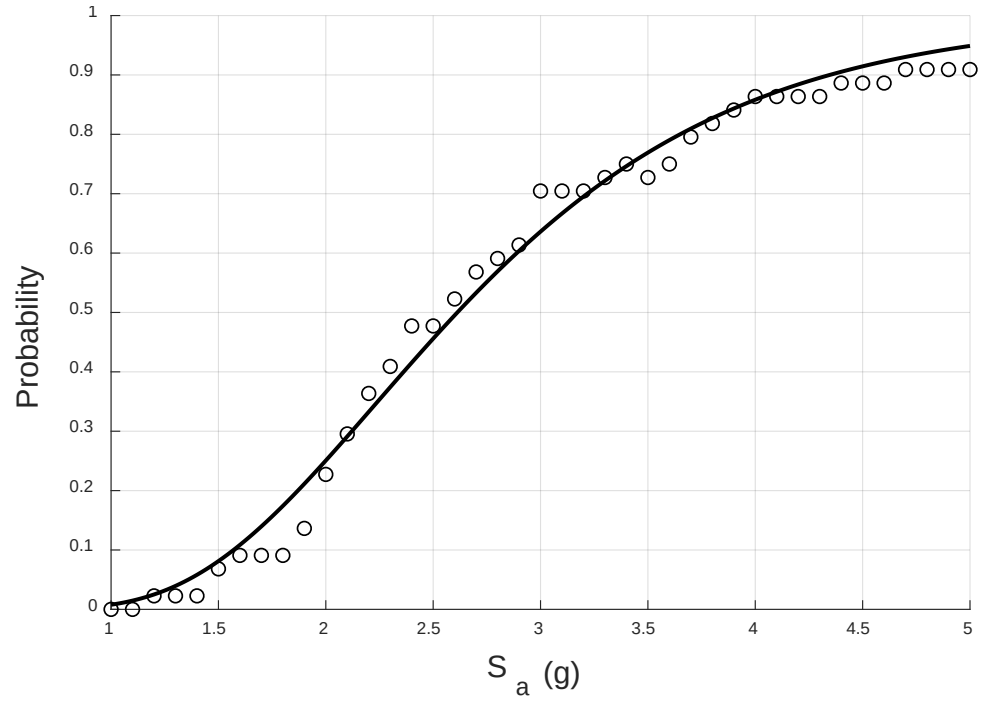


Figure I124. Archetype 123 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

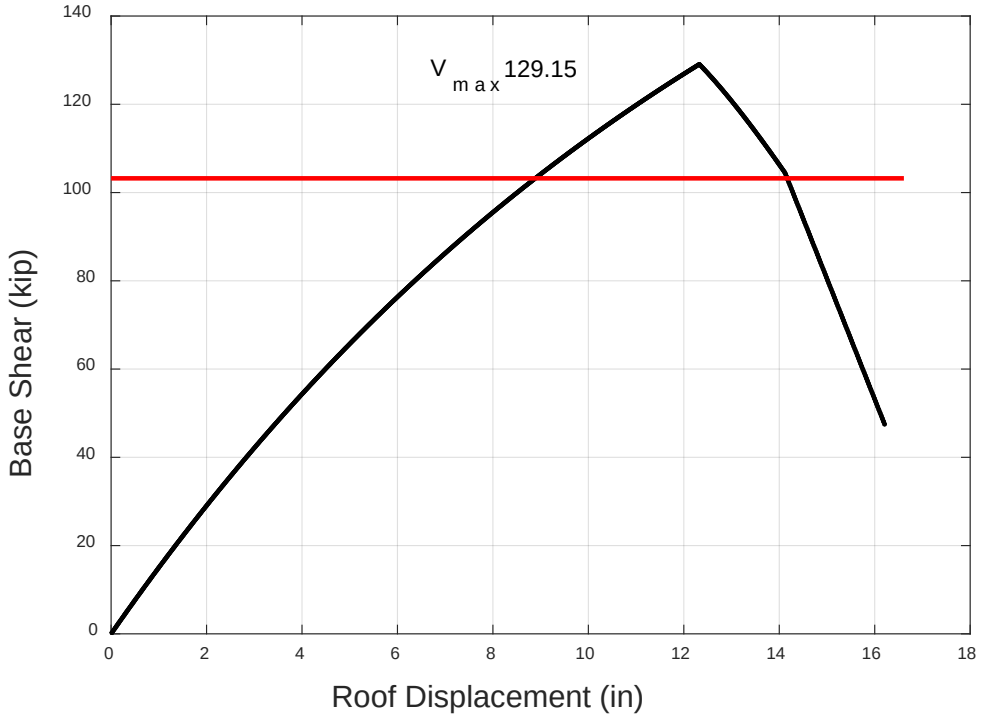


Figure I125. Archetype 124, R = 3, static pushover.

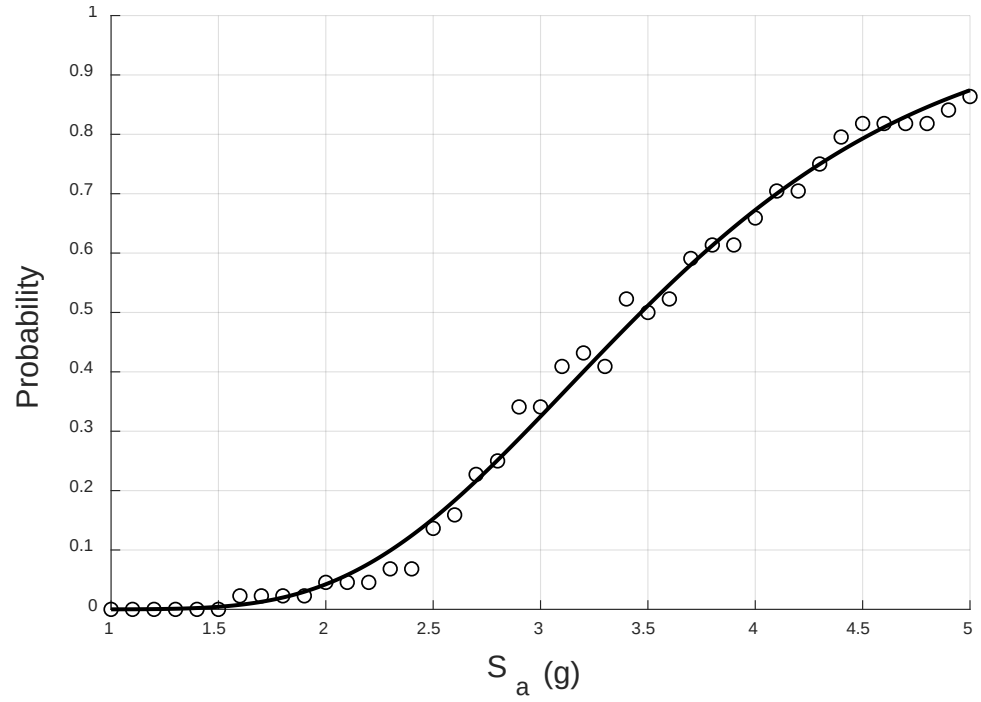


Figure I126. Archetype 124 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

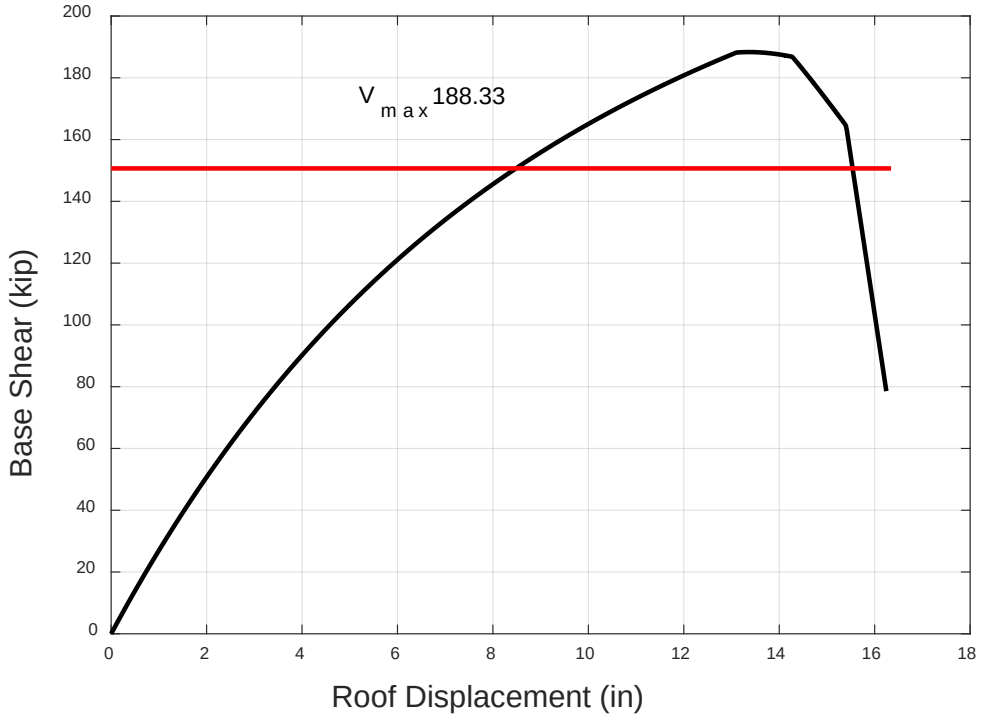


Figure I127. Archetype 125, R = 3, static pushover.

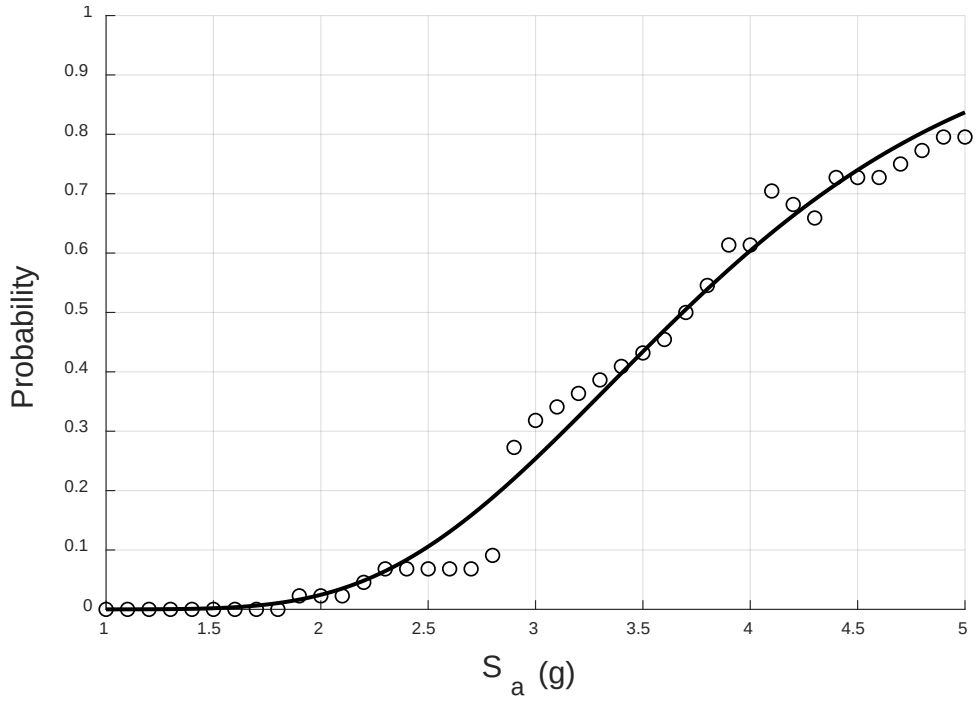


Figure I128. Archetype 125 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

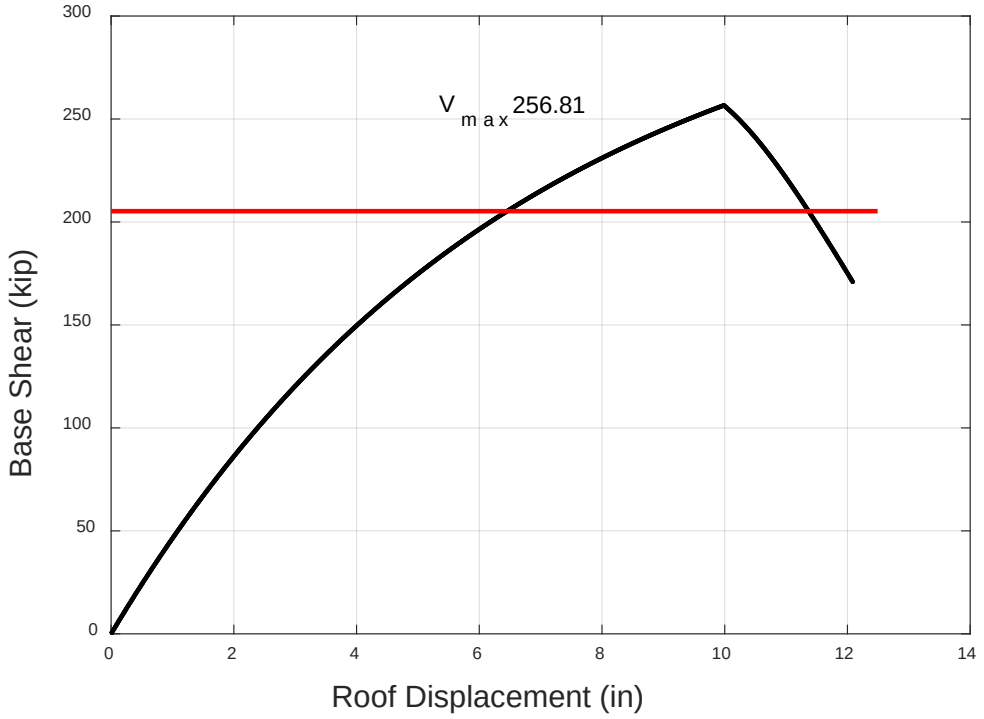


Figure I129. Archetype 126, R = 3, static pushover.

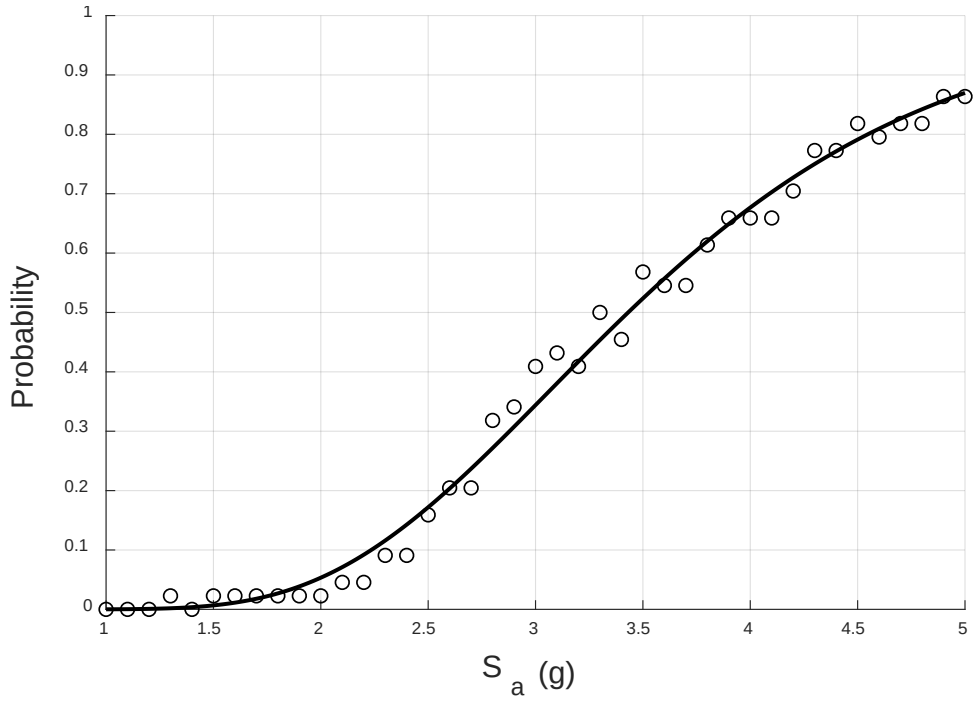


Figure I130. Archetype 126 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

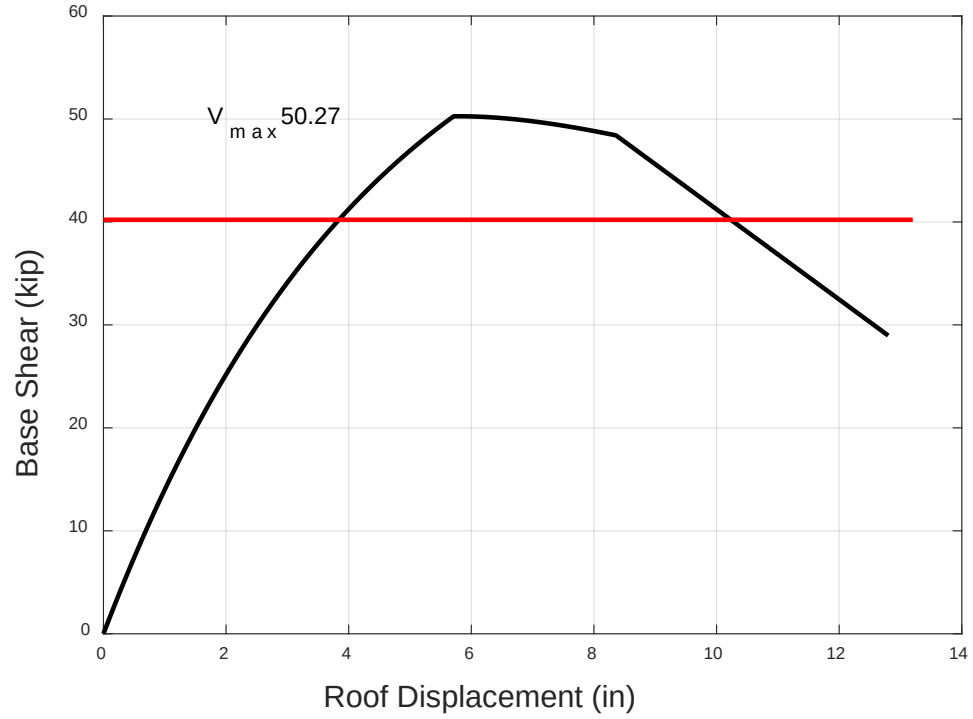


Figure I131. Archetype 133, R = 3, static pushover.

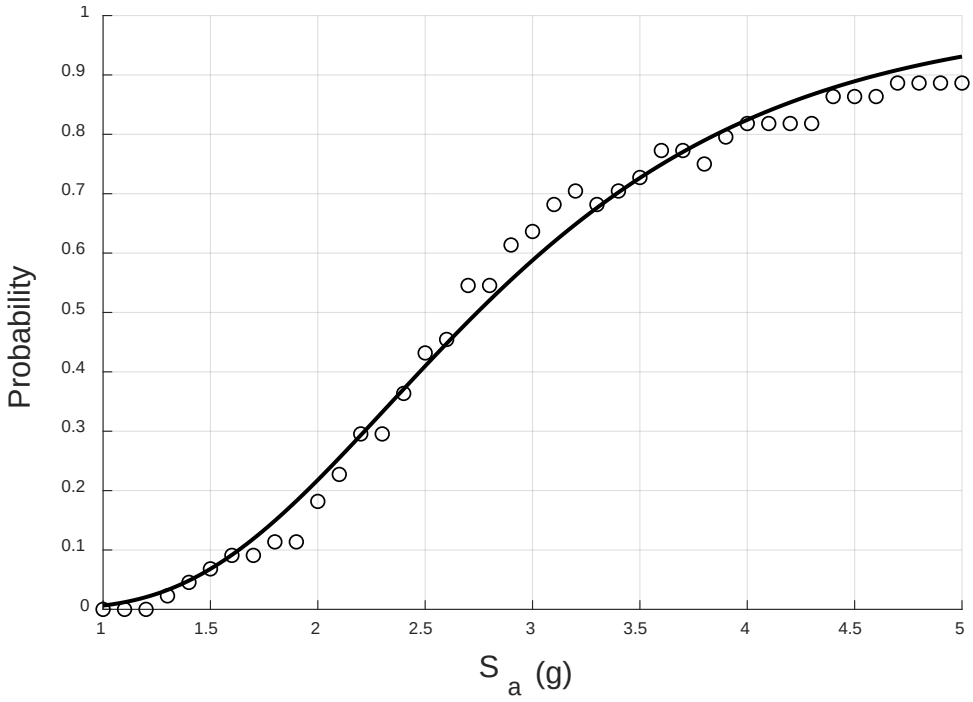


Figure I132. Archetype 133 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

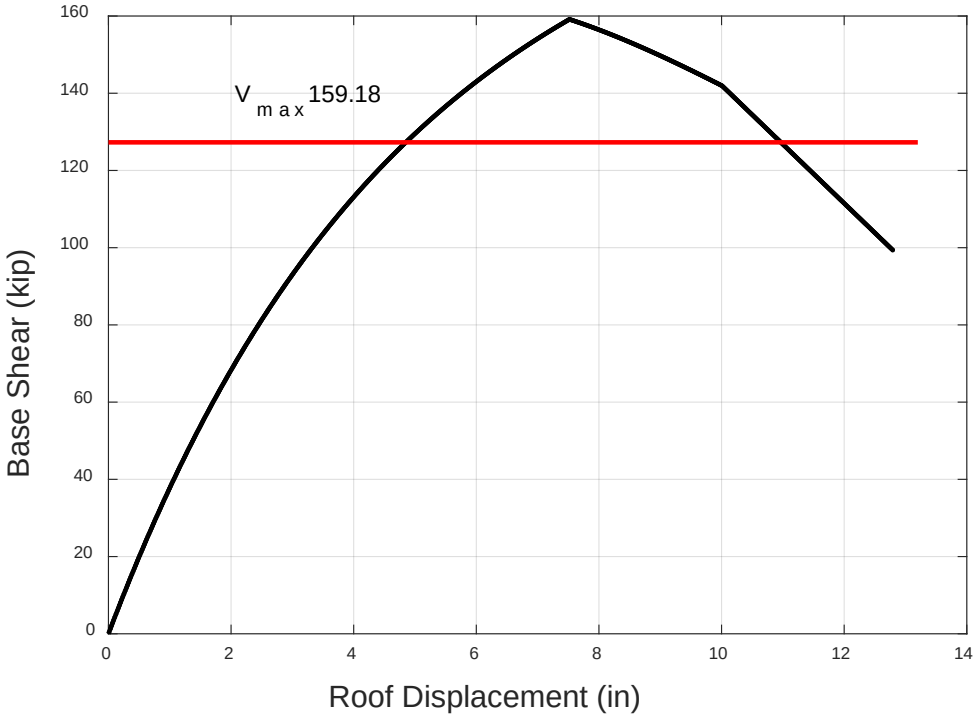


Figure I133. Archetype 135, R = 3, static pushover.

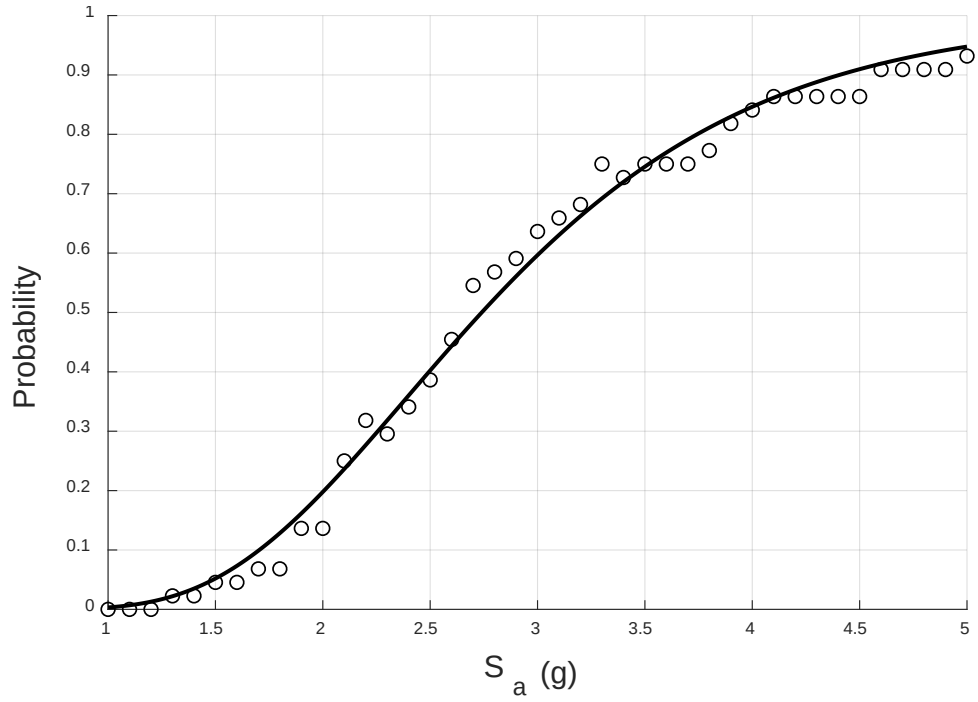


Figure I134. Archetype 135 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

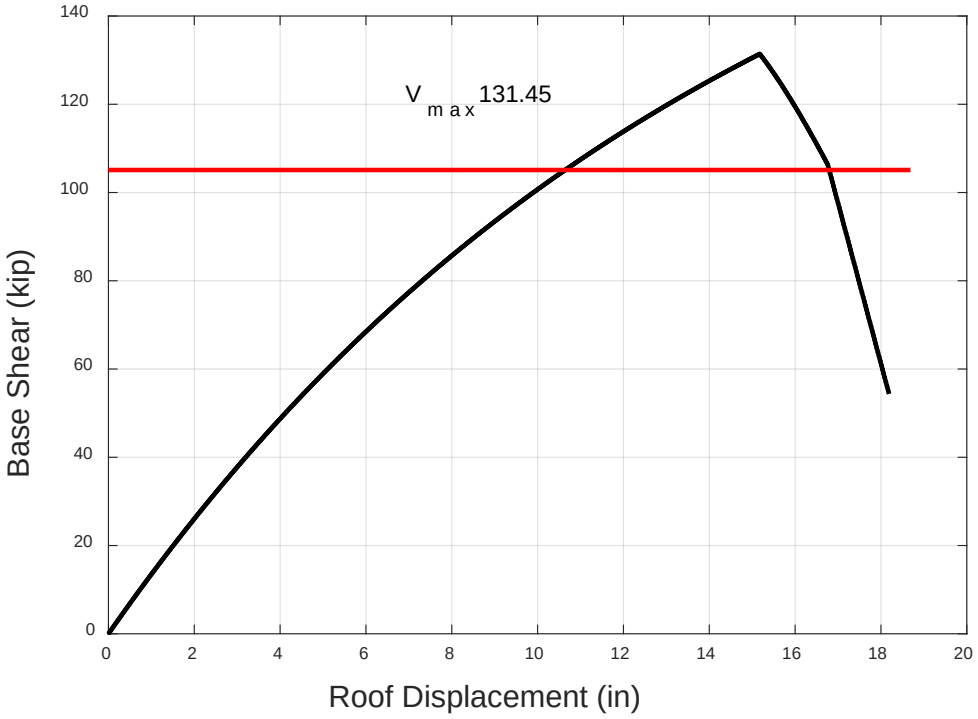


Figure I135. Archetype 136, R = 3, static pushover.

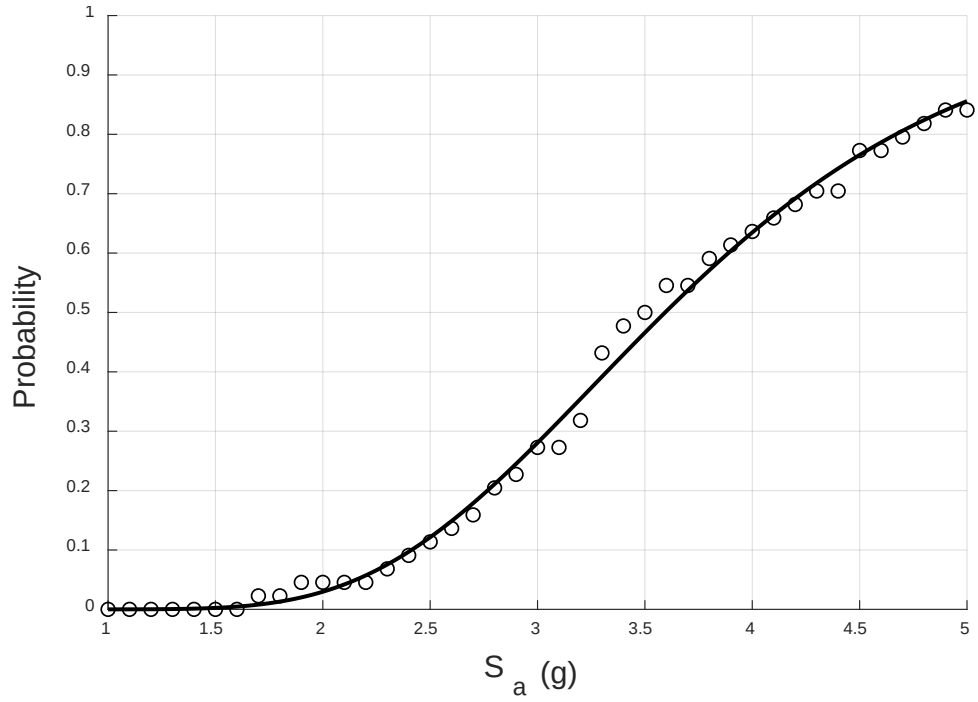


Figure I136. Archetype 136 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

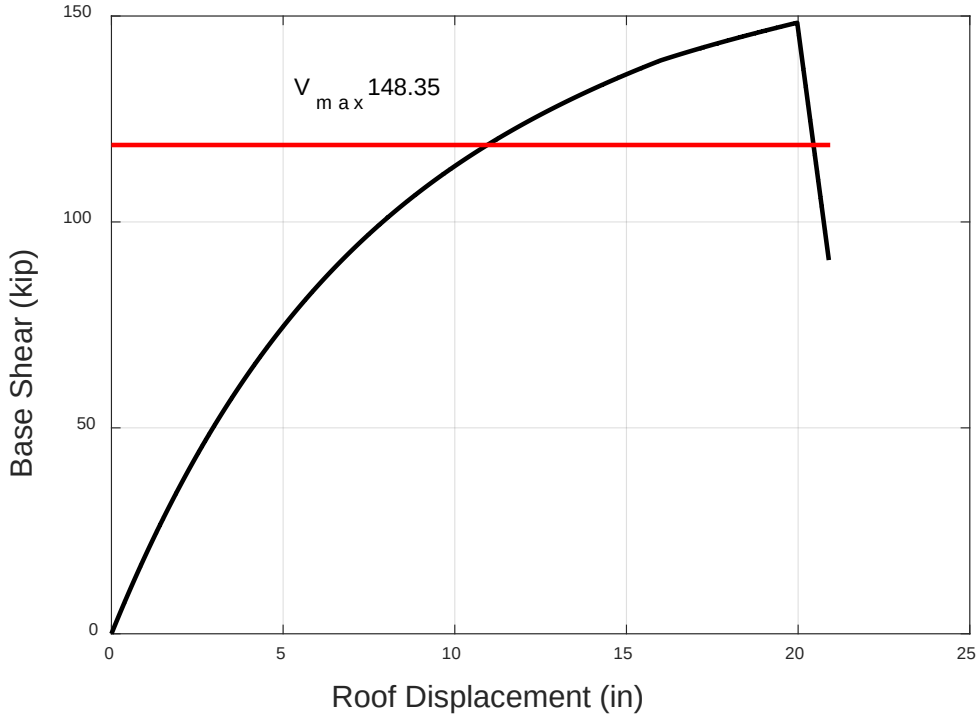


Figure I137. Archetype 137, R = 3, static pushover.

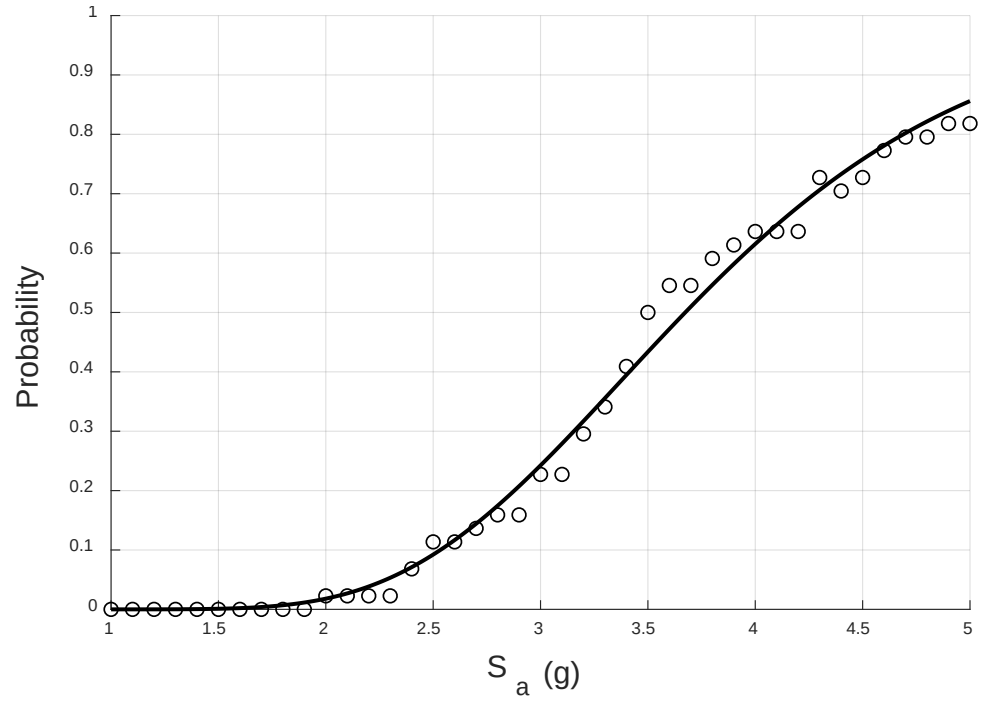


Figure I138. Archetype 137 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

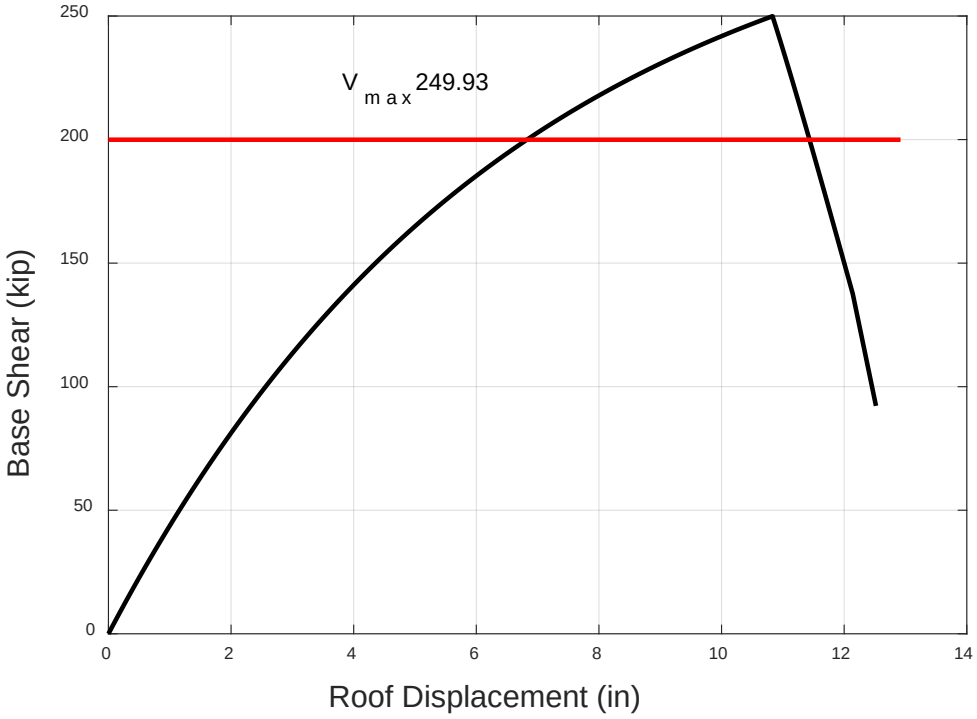


Figure I139. Archetype 138, R = 3, static pushover.

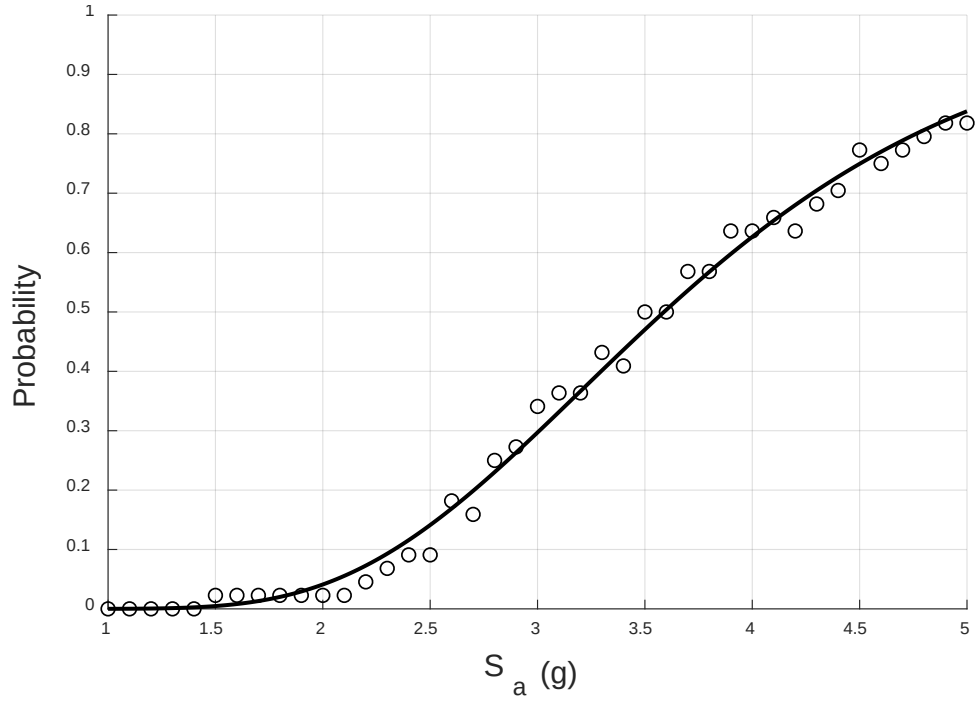


Figure I140. Archetype 138 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit.

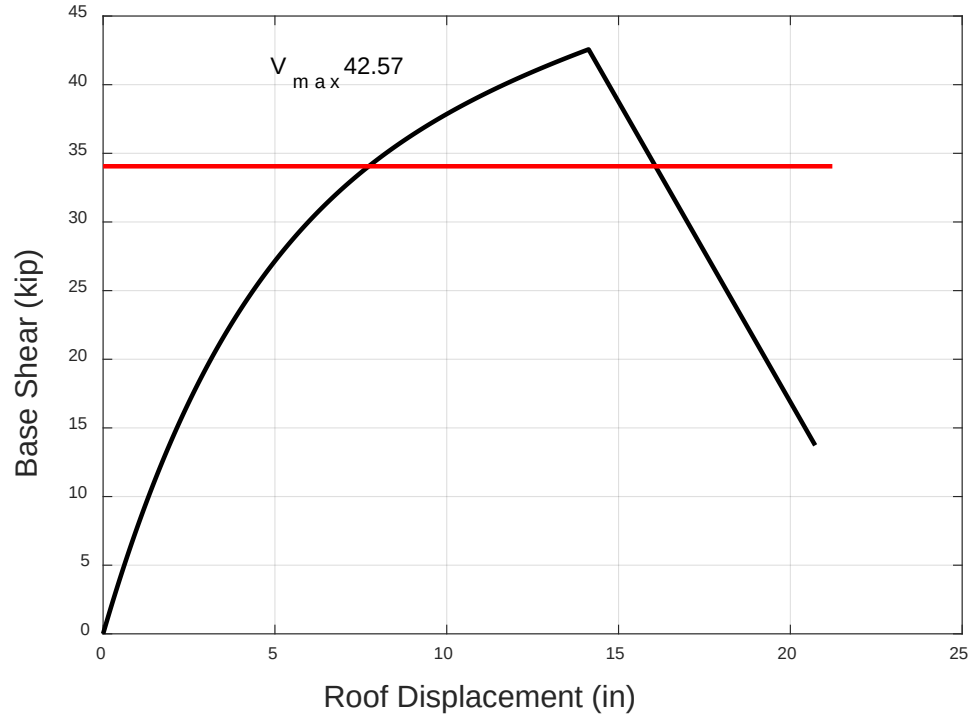


Figure I141. Archetype 25, R = 4, static pushover.

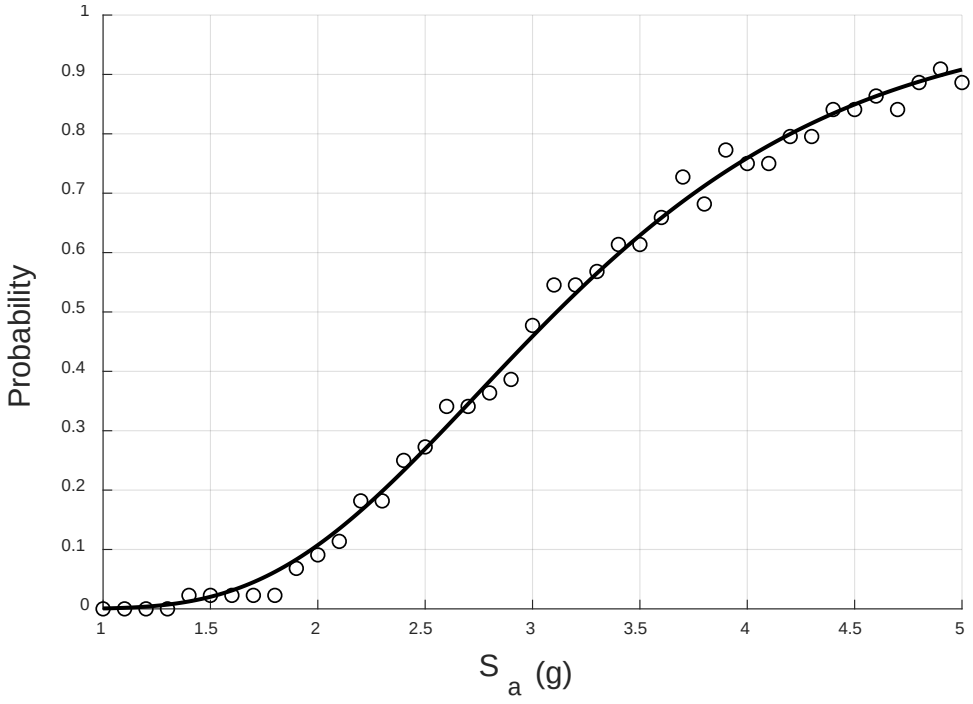


Figure I142. Archetype 25 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

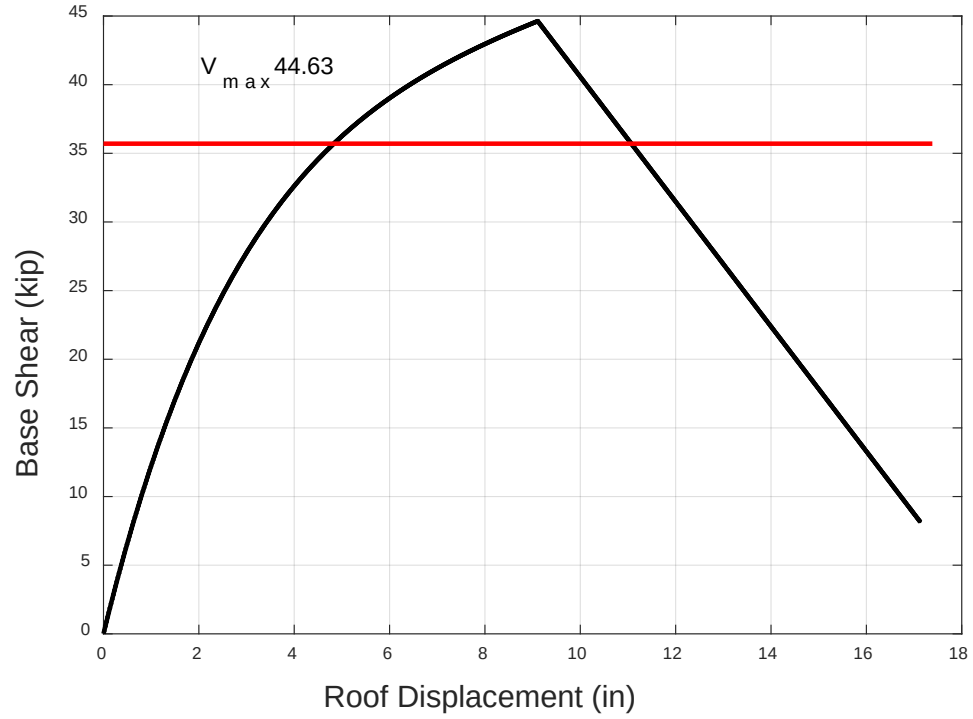


Figure I143. Archetype 26, R = 4, static pushover.

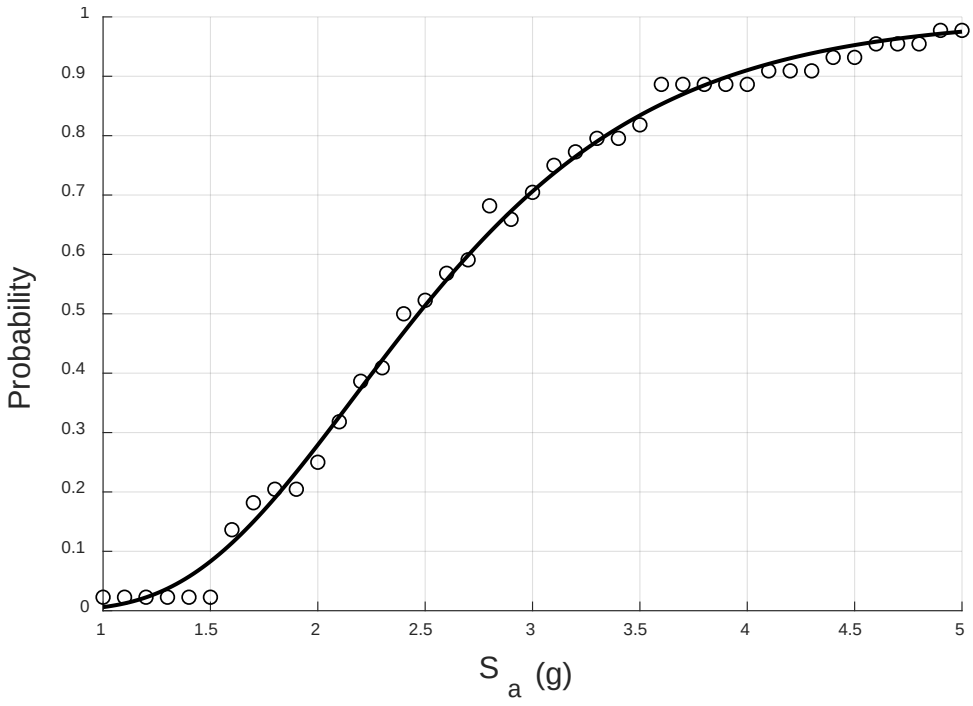


Figure I144. Archetype 26 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

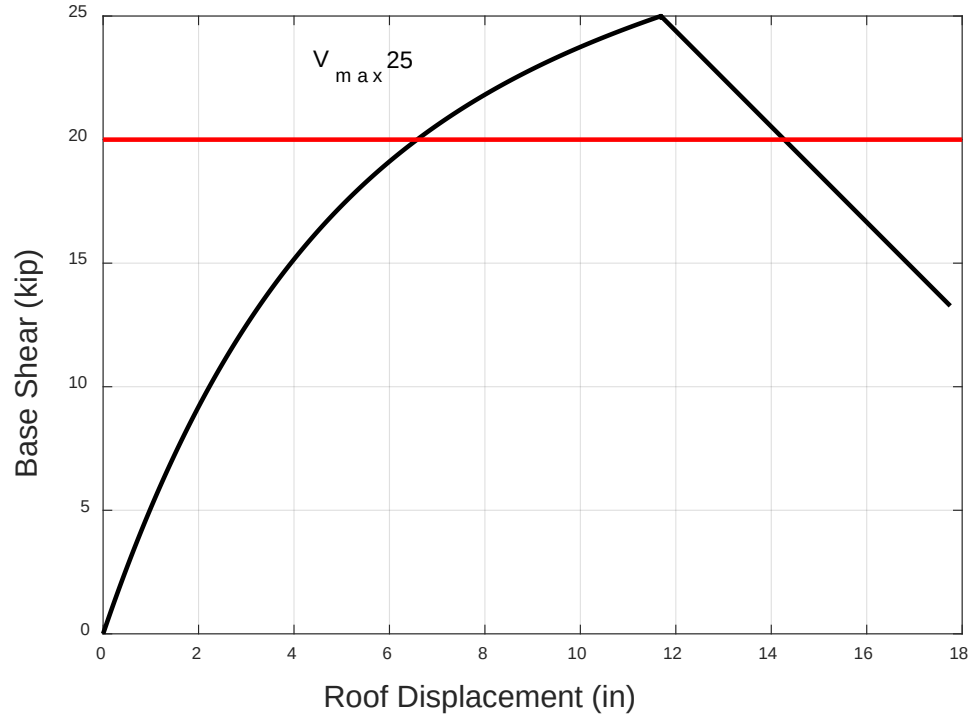


Figure I145. Archetype 27, R = 4, static pushover.

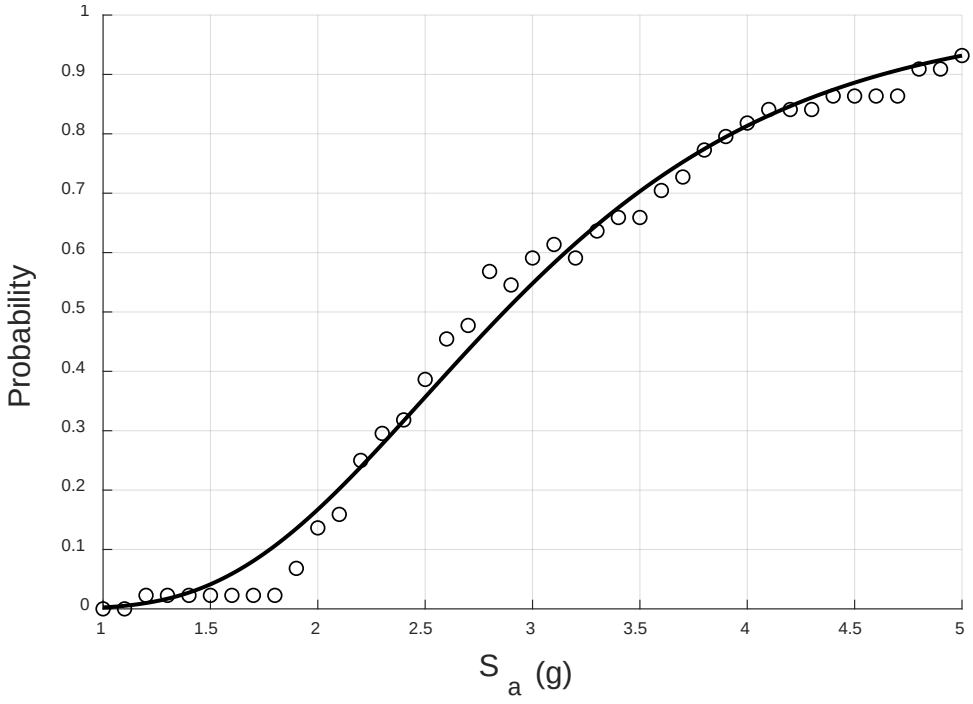


Figure I146. Archetype 27 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

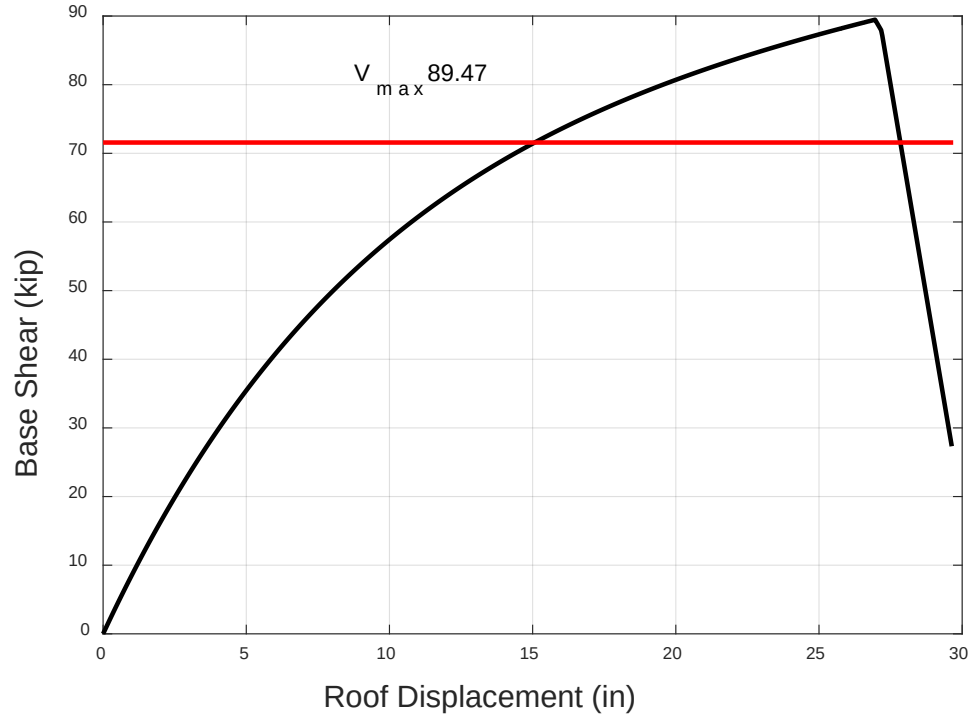


Figure I147. Archetype 28, R = 4, static pushover.

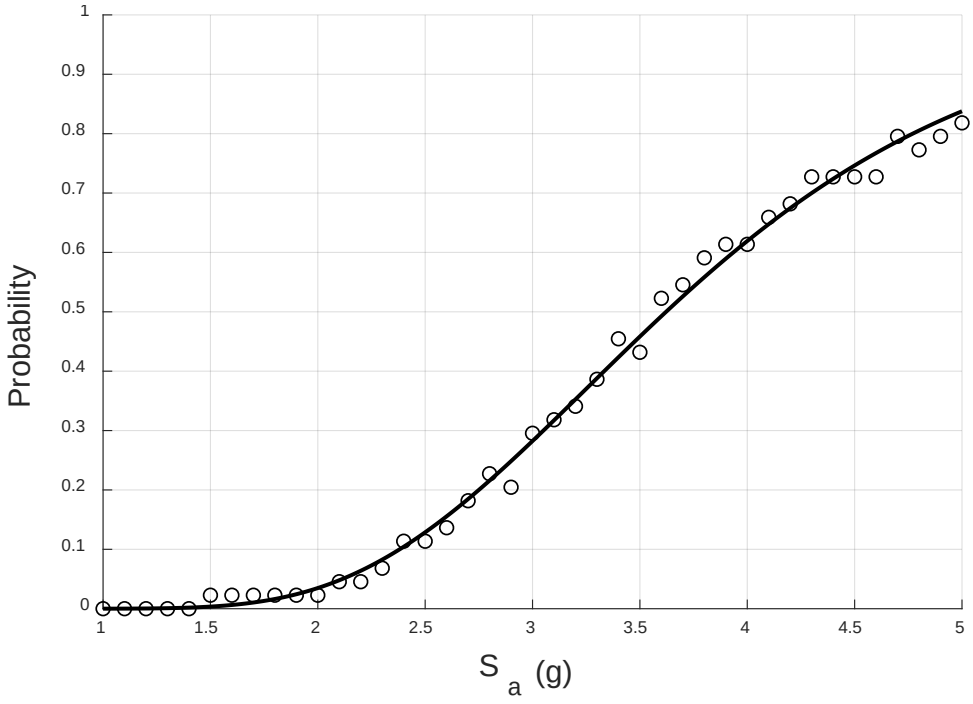


Figure I148. Archetype 28 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

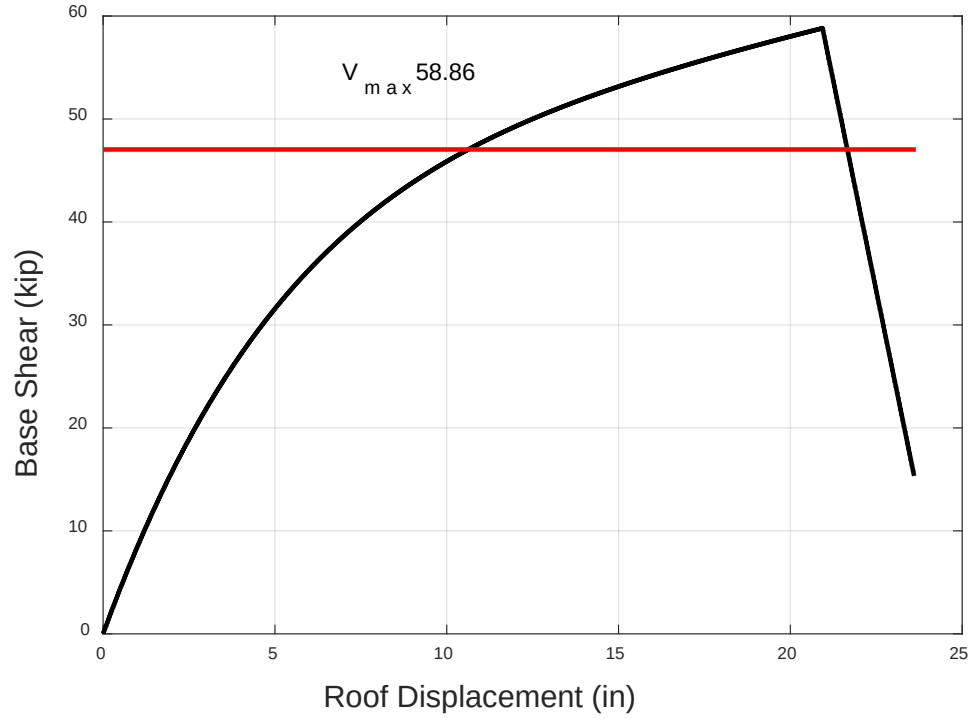


Figure I149. Archetype 29, R = 4, static pushover.

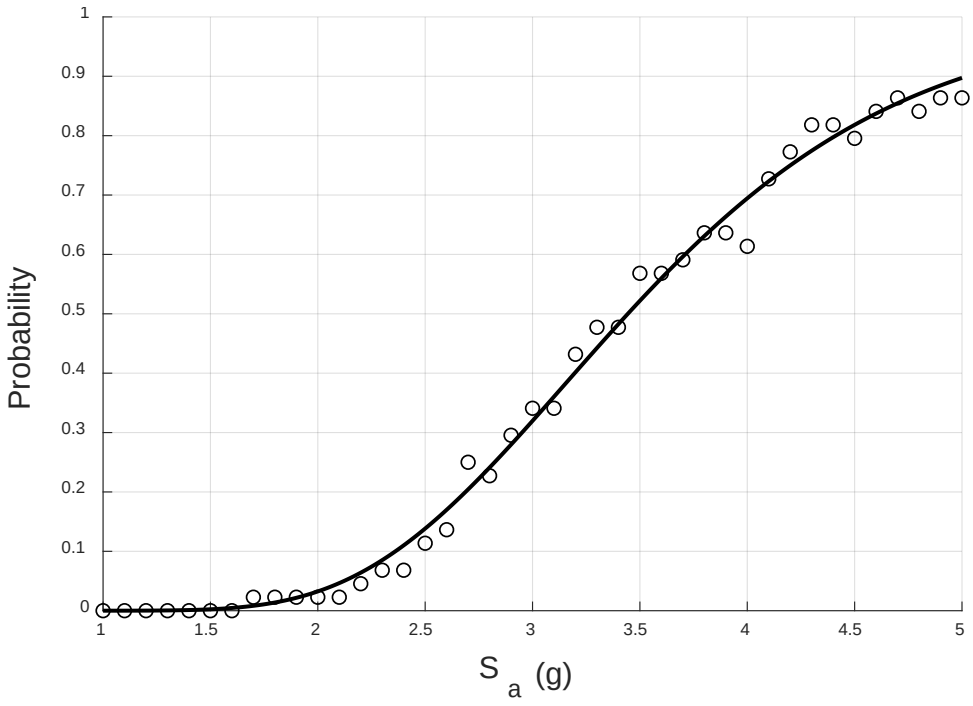


Figure I150. Archetype 29 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

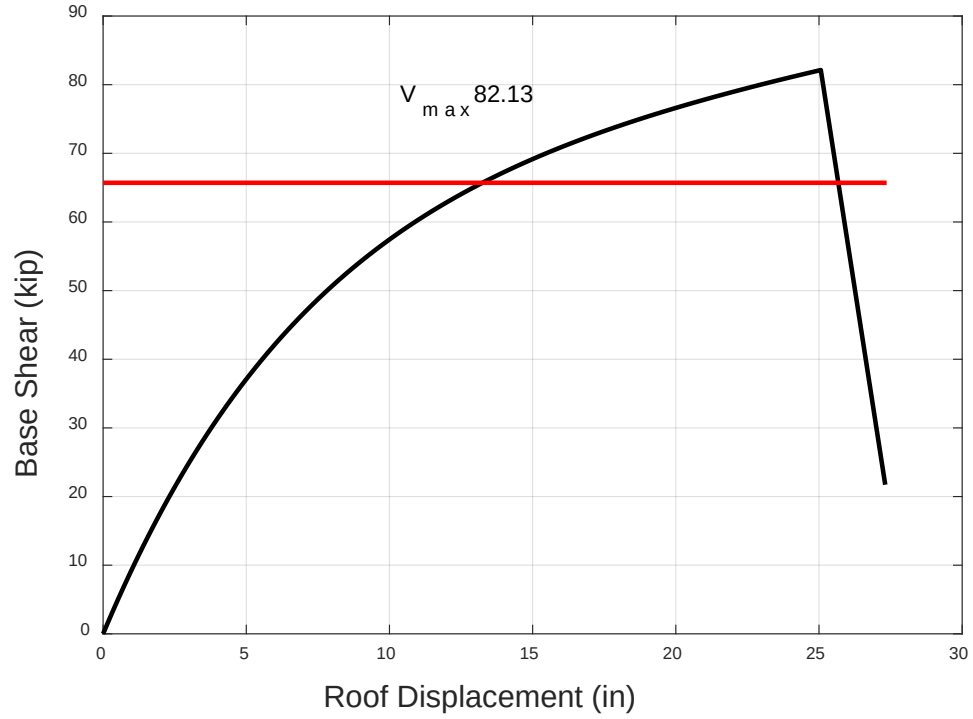


Figure I151. Archetype 30, R = 4, static pushover.

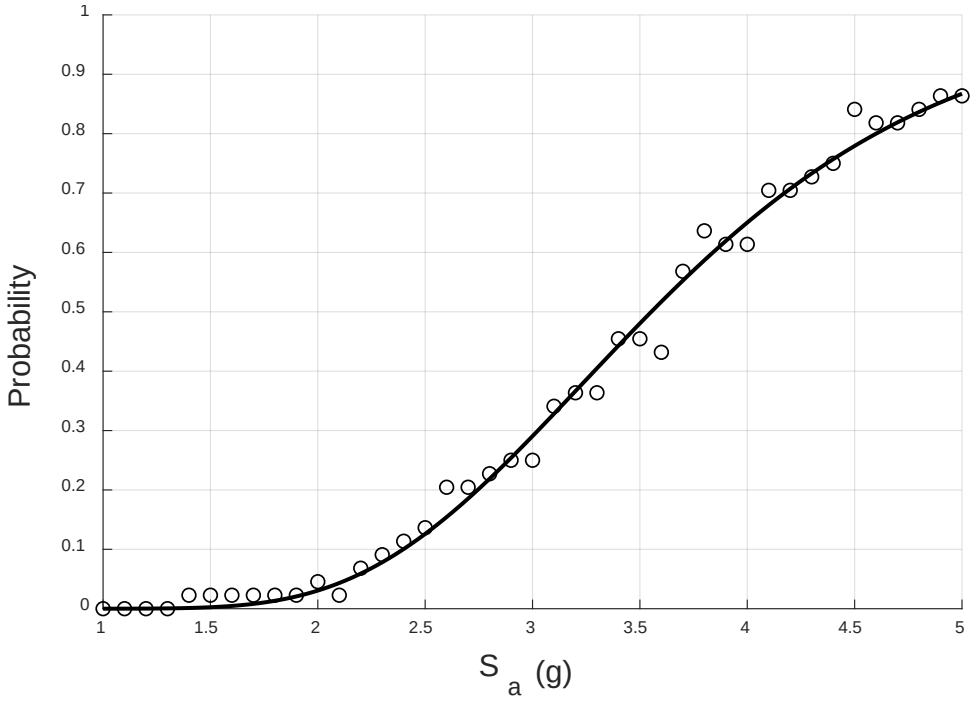


Figure I152. Archetype 30 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

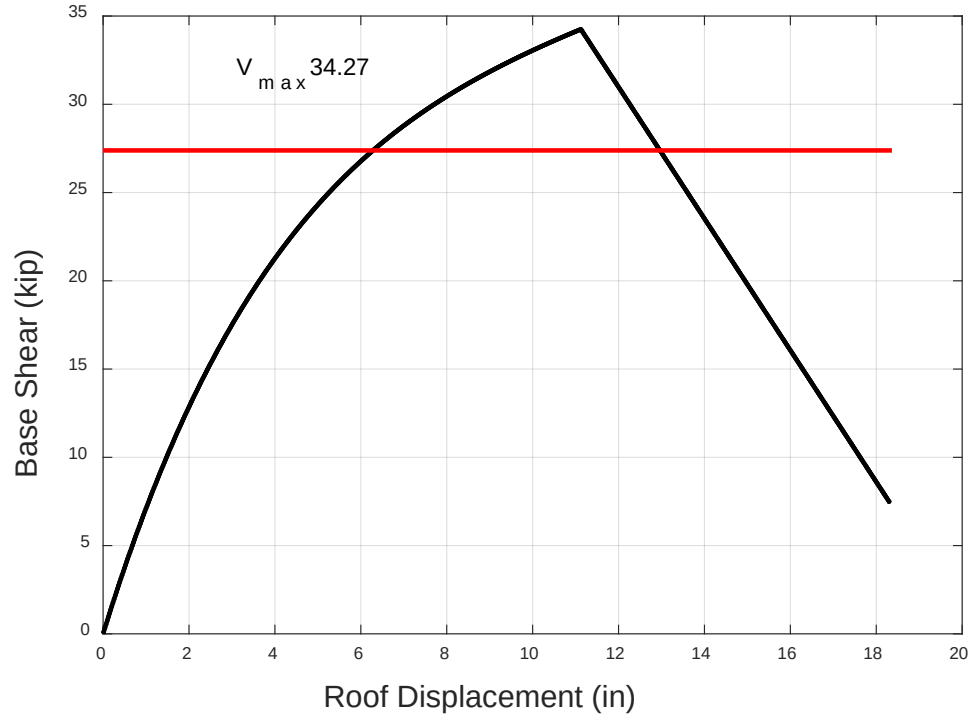


Figure I153. Archetype 37, R = 4, static pushover.

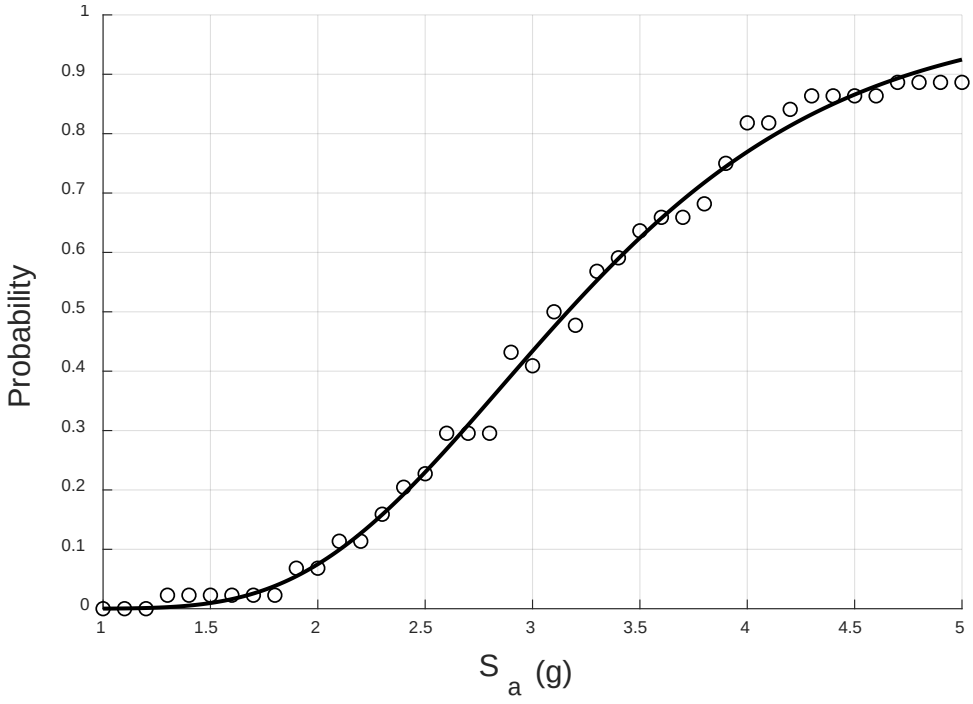


Figure I154. Archetype 37 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

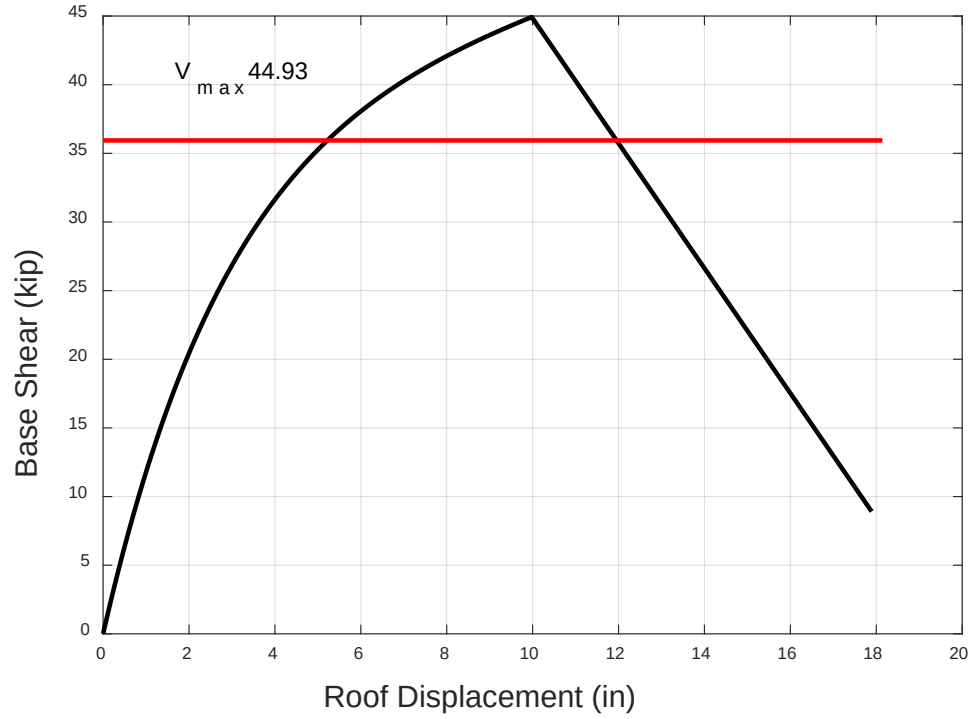


Figure I155. Archetype 38, R = 4, static pushover.

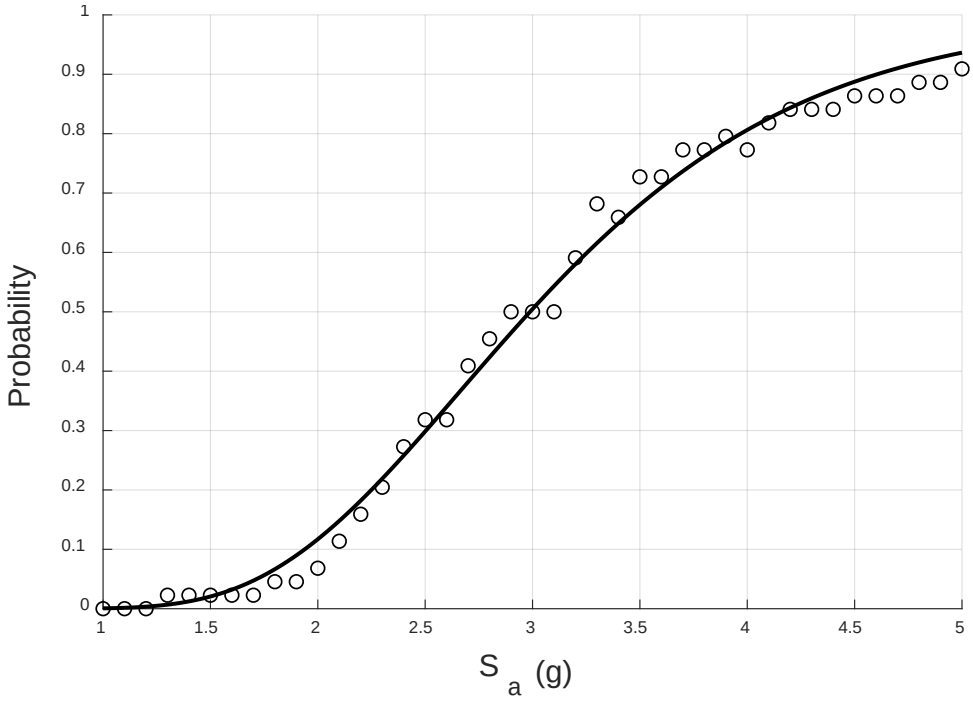


Figure I156. Archetype 38 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

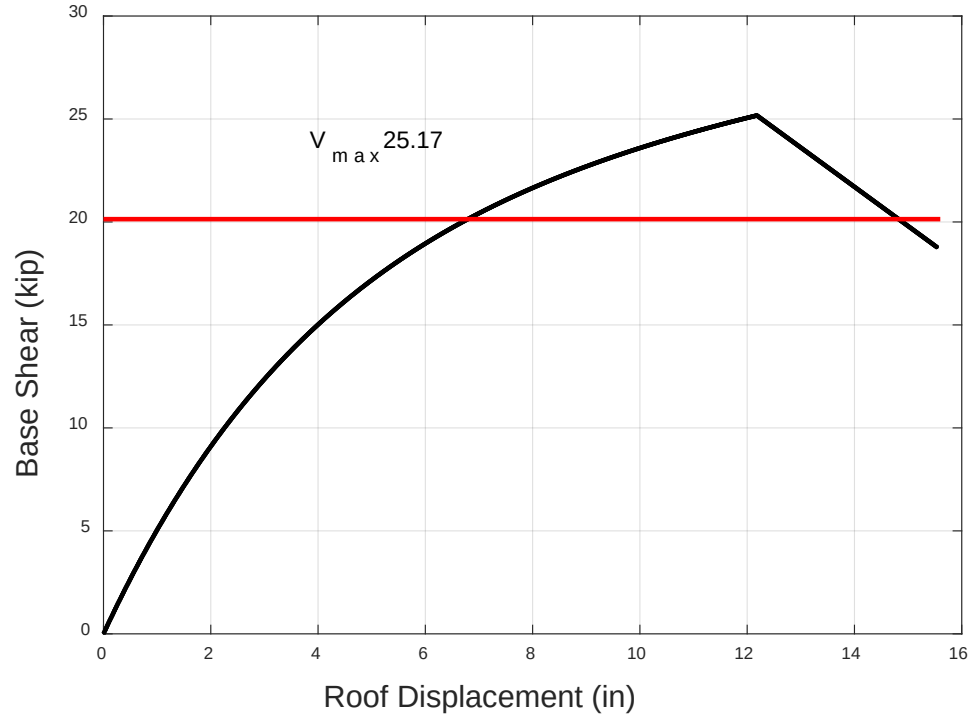


Figure I157. Archetype 39, R = 4, static pushover.

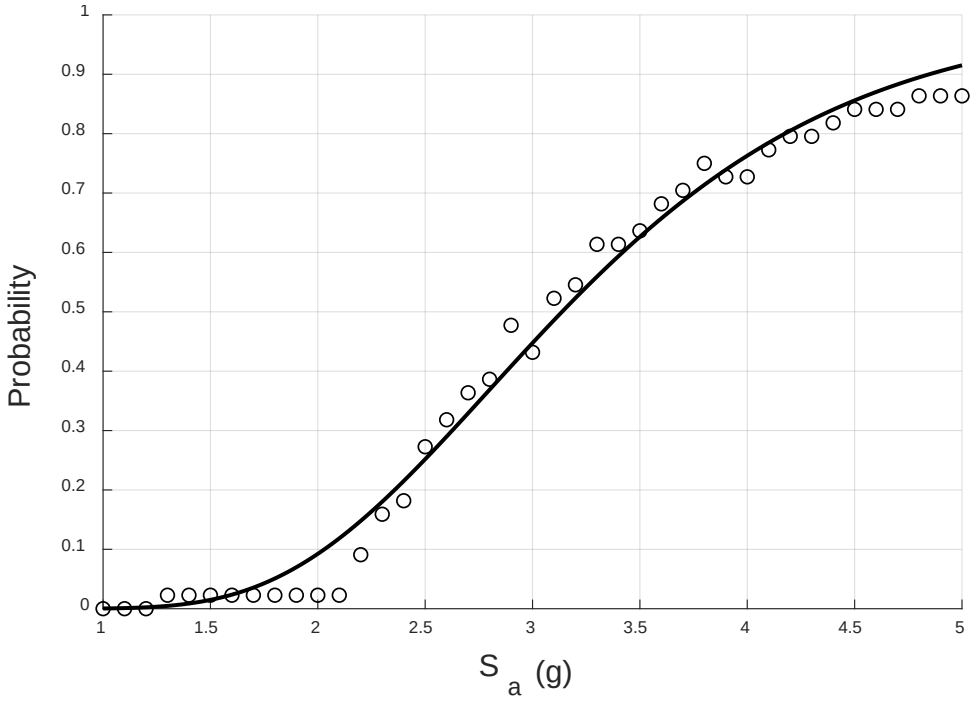


Figure I158. Archetype 39 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

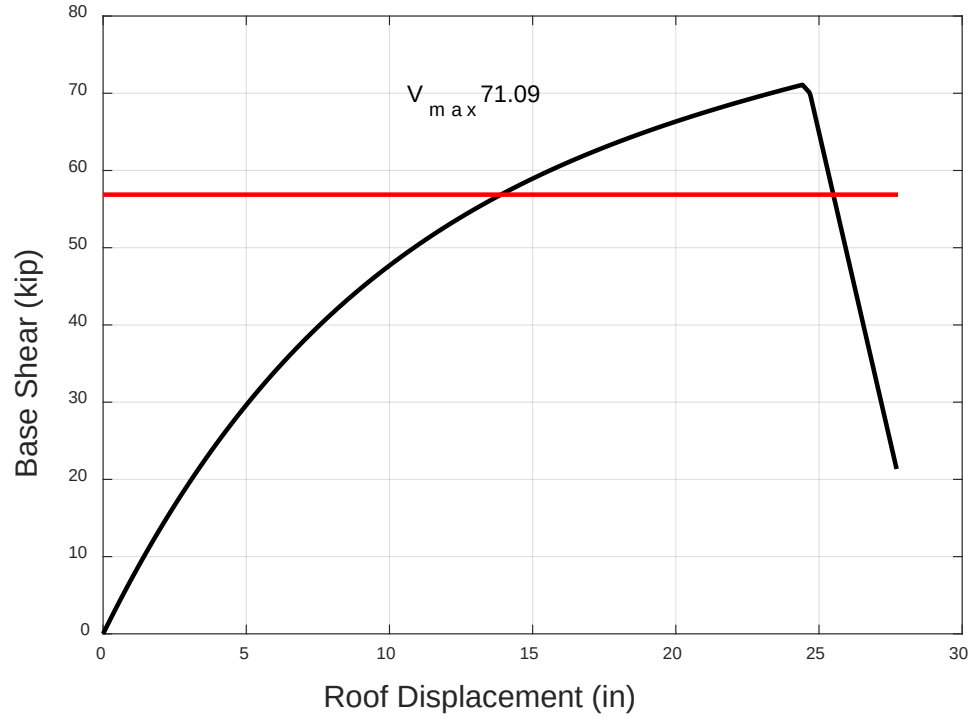


Figure I159. Archetype 40, R = 4, static pushover.

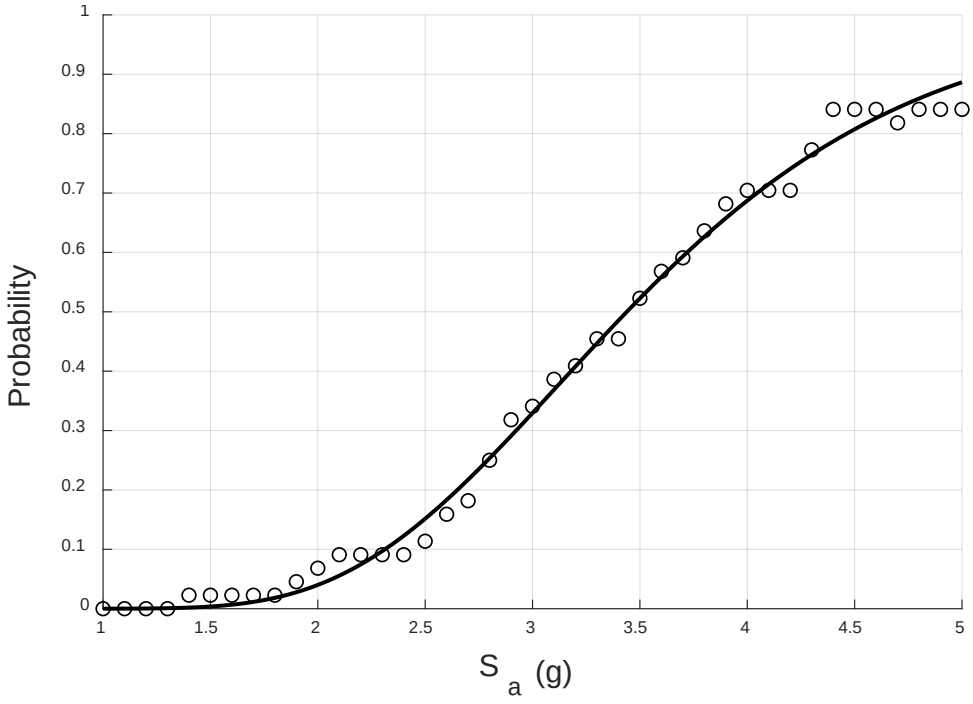


Figure I160. Archetype 40 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

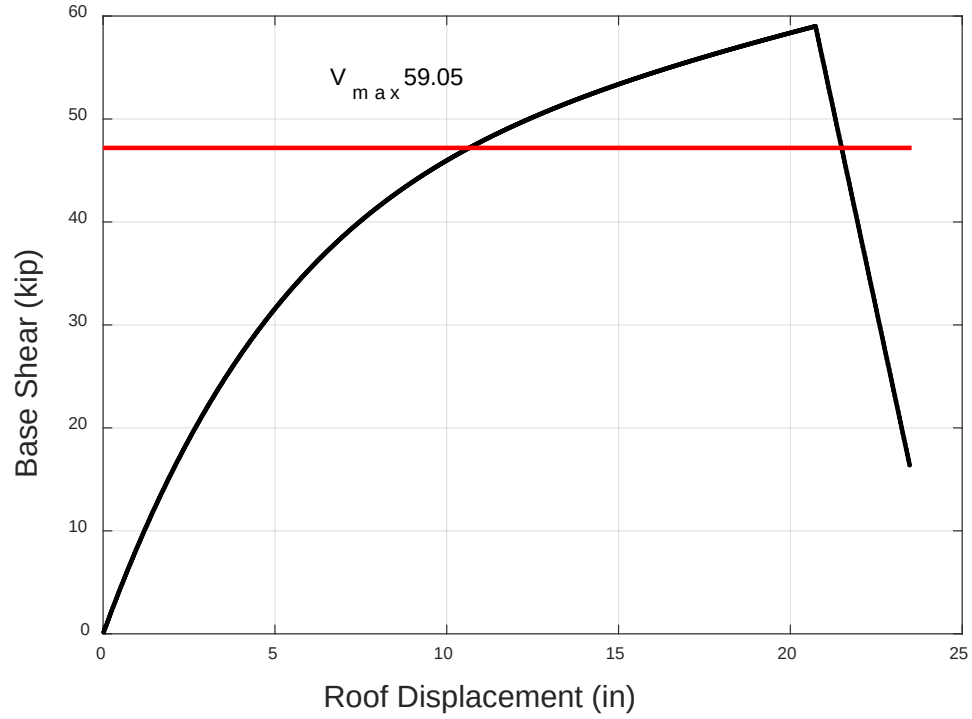


Figure I161. Archetype 41, R = 4, static pushover.

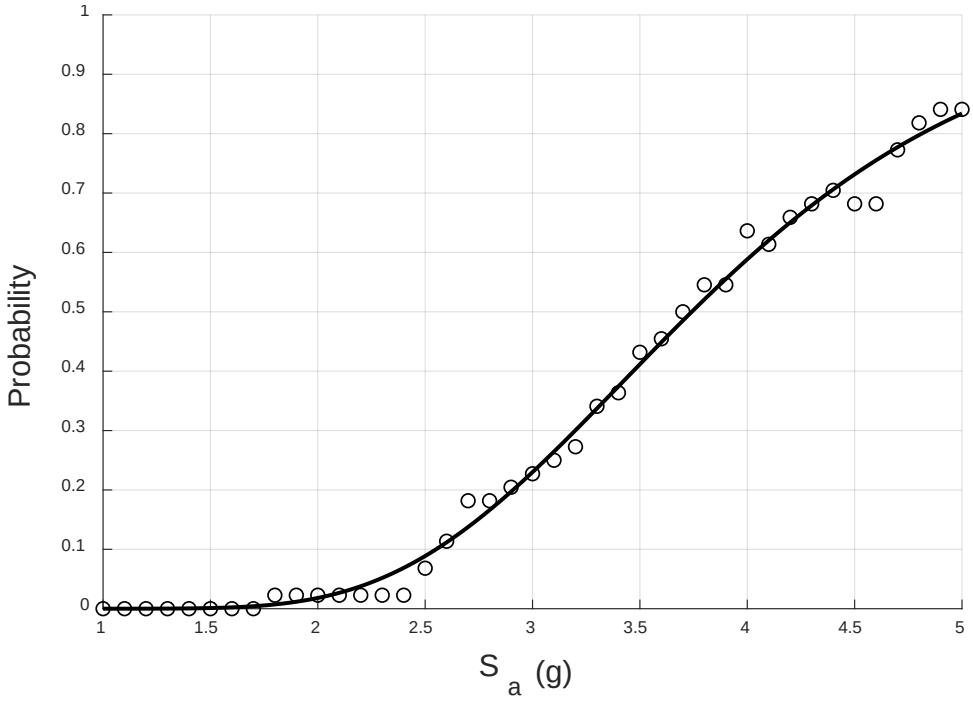


Figure I162. Archetype 41 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

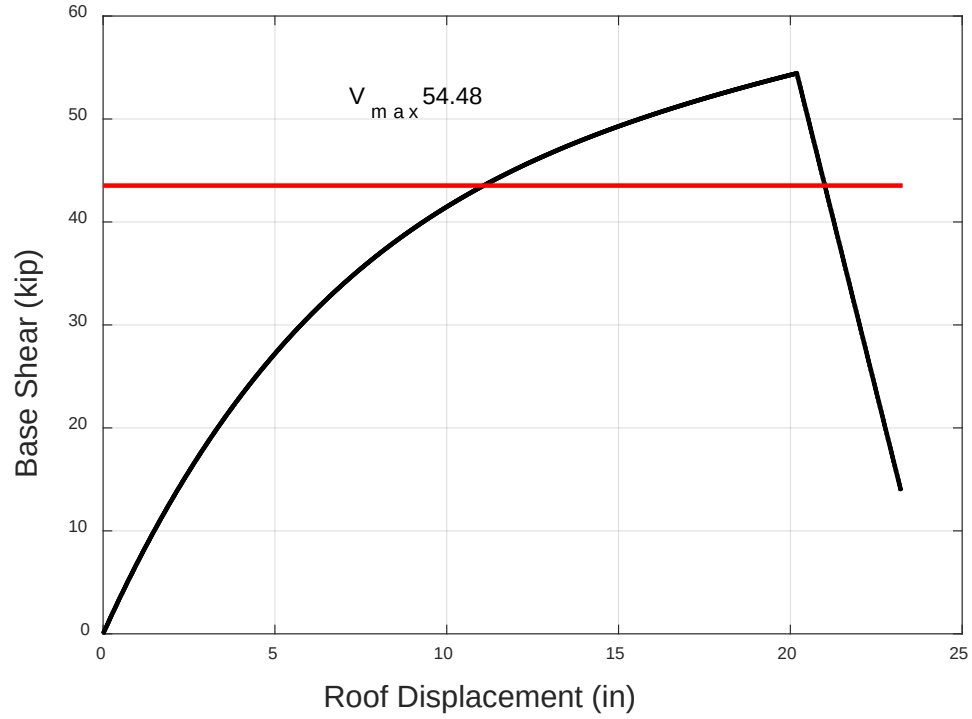


Figure I163. Archetype 42, R = 4, static pushover.

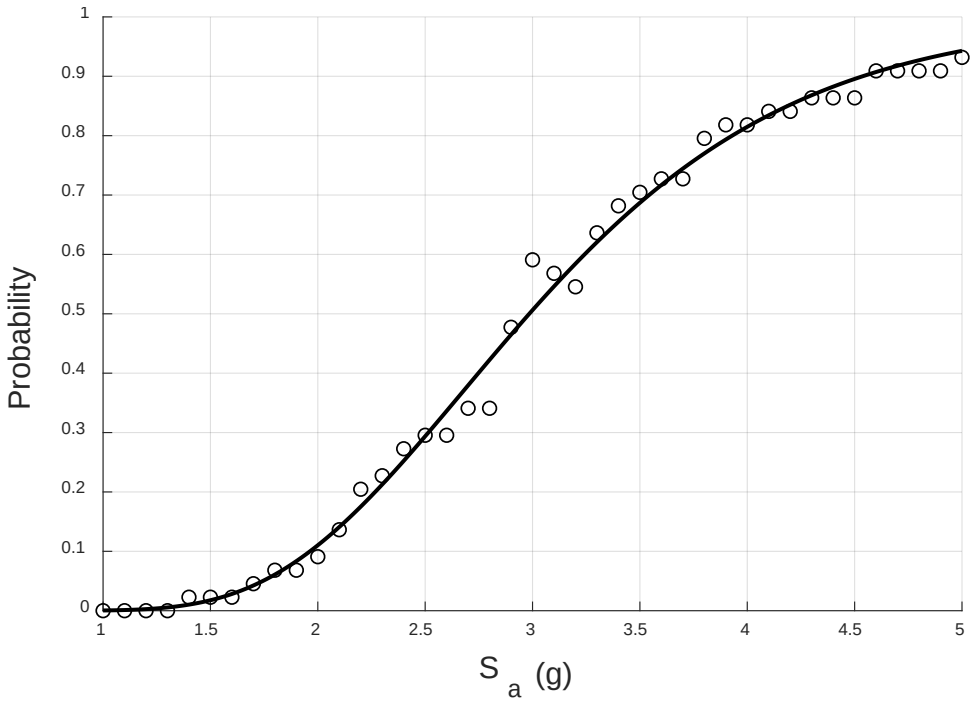


Figure I164. Archetype 42 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

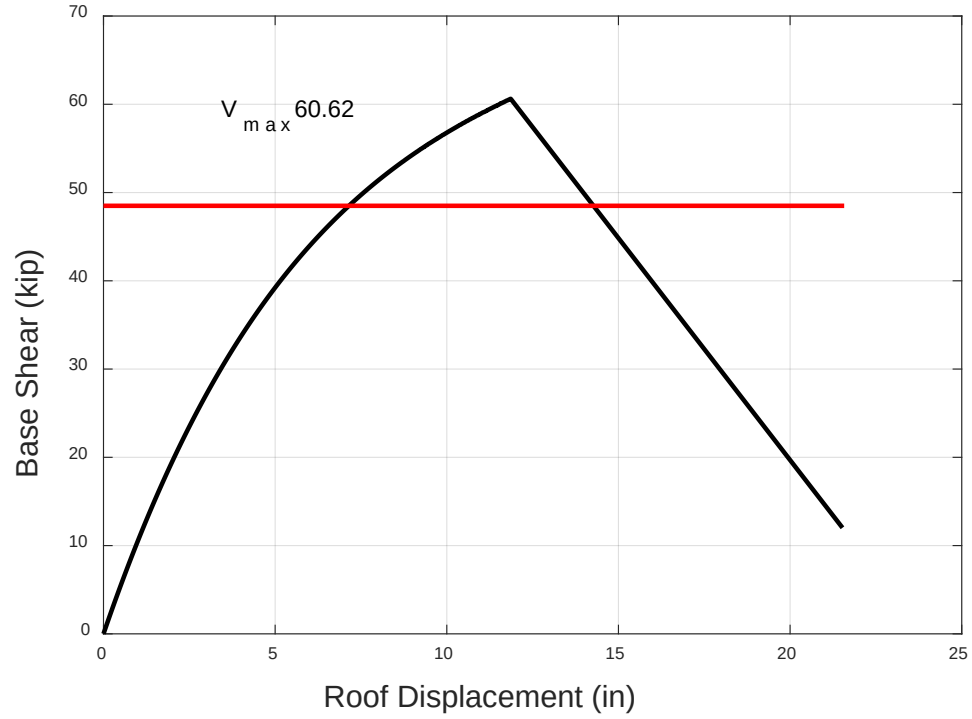


Figure I165. Archetype 97, R = 4, static pushover.

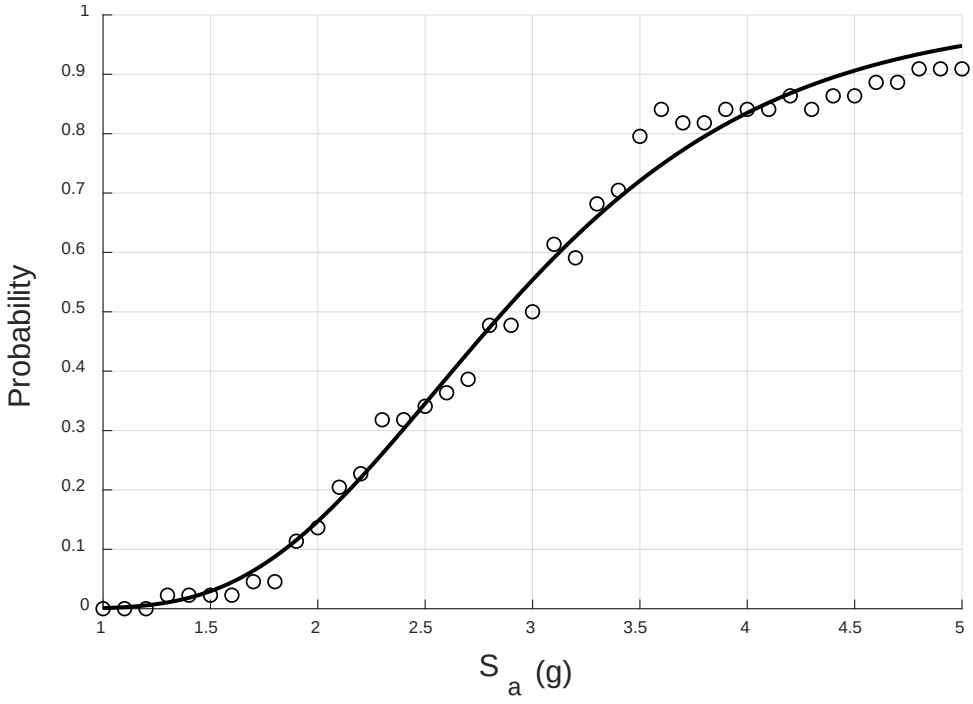


Figure I166. Archetype 97 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

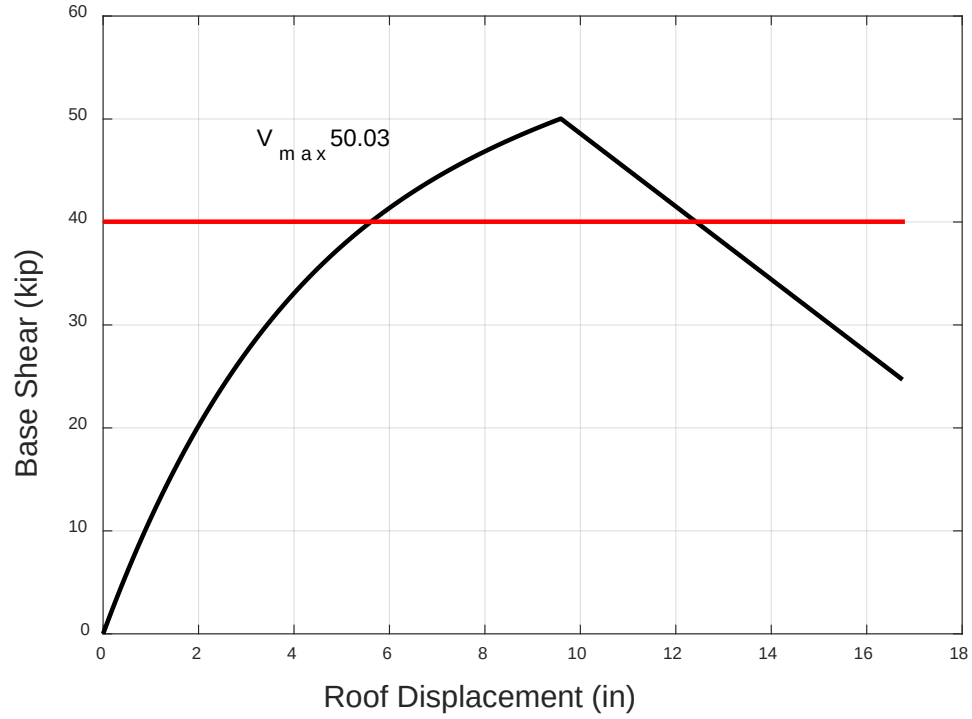


Figure I167. Archetype 98, R = 4, static pushover.

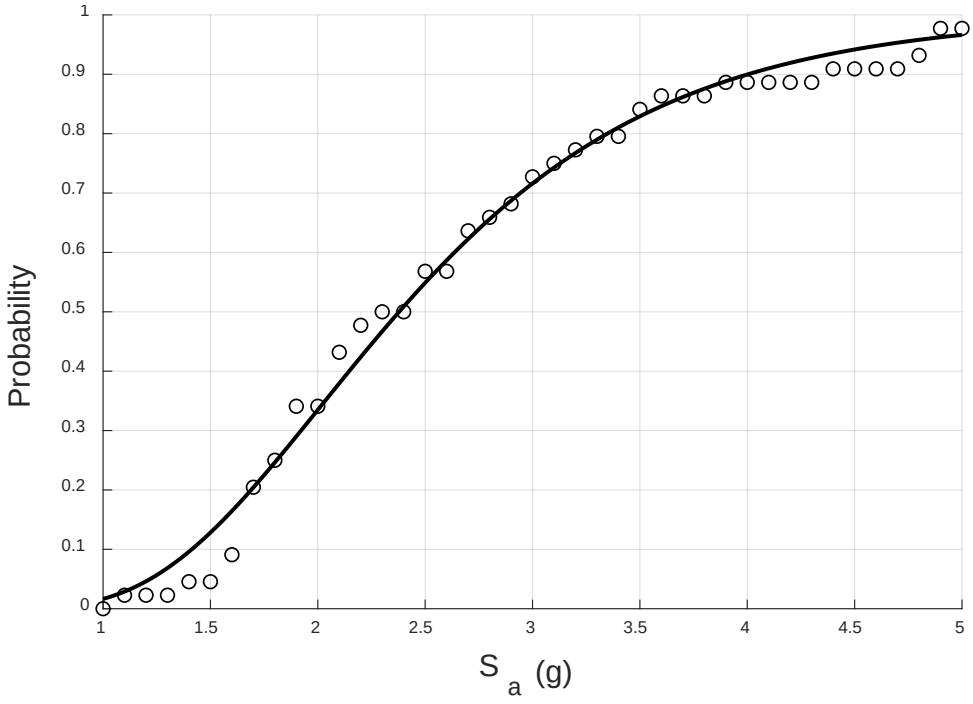


Figure I168. Archetype 98 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

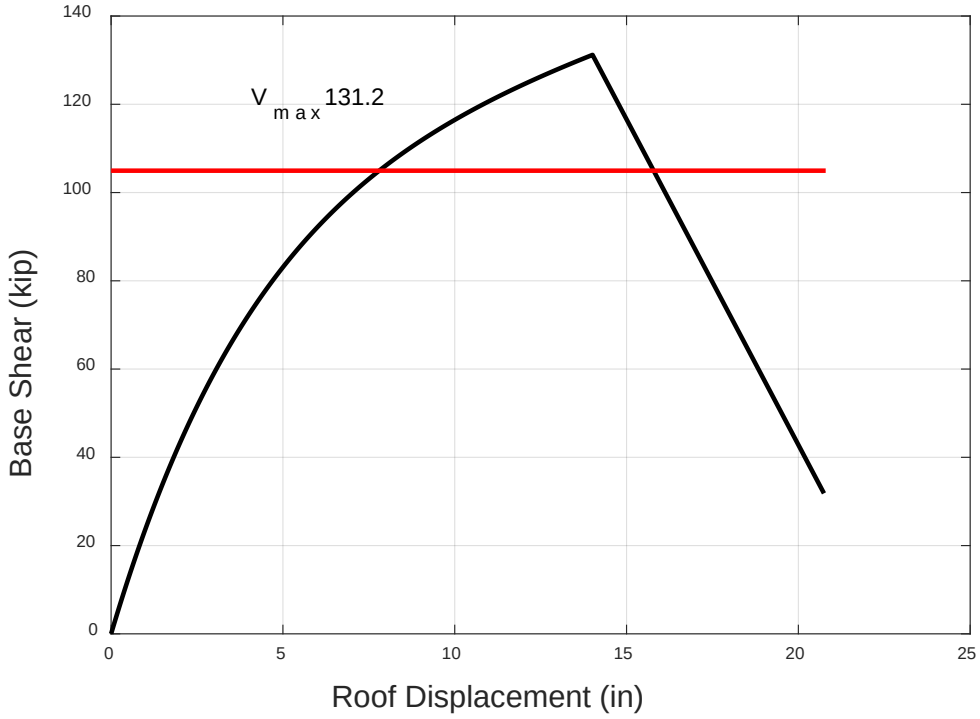


Figure I169. Archetype 99, R = 4, static pushover.

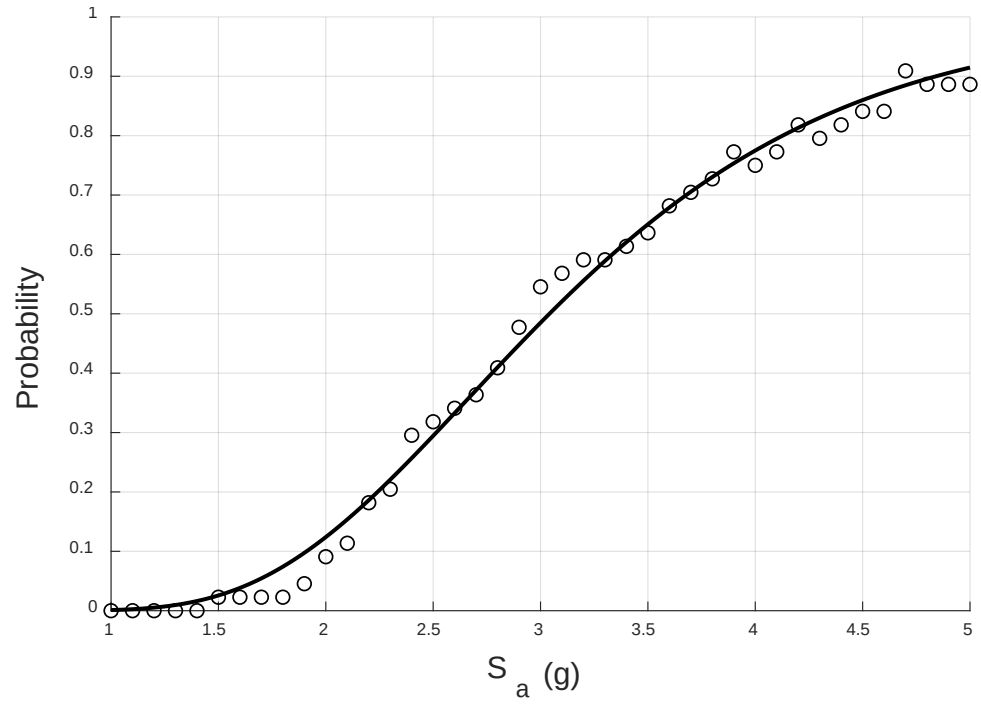


Figure I170. Archetype 99 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

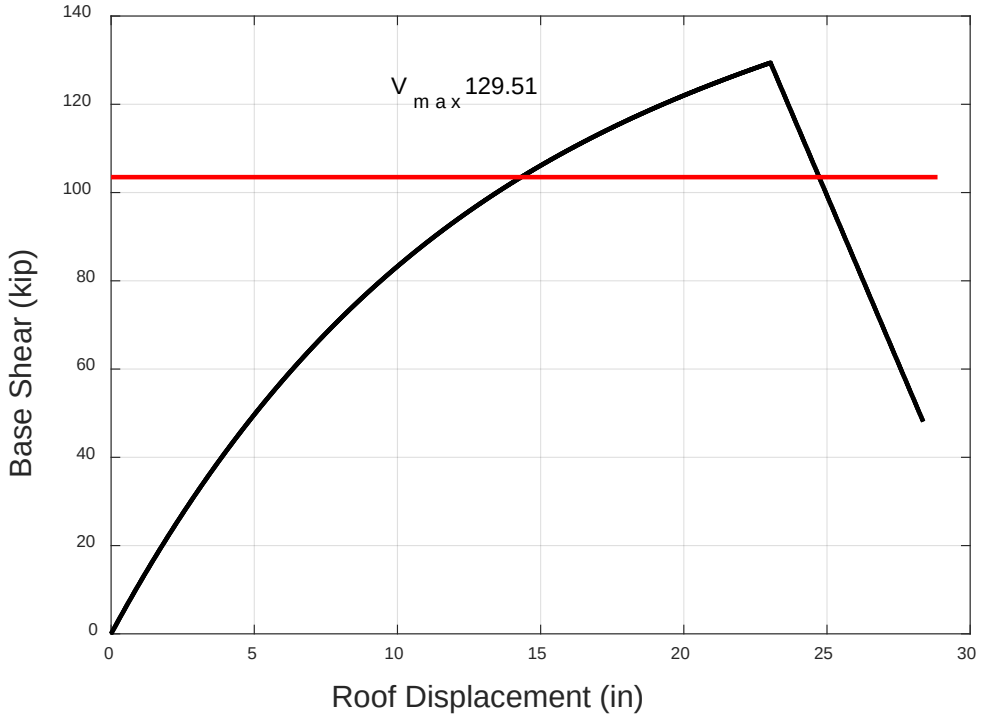


Figure I171. Archetype 100, R = 4, static pushover.

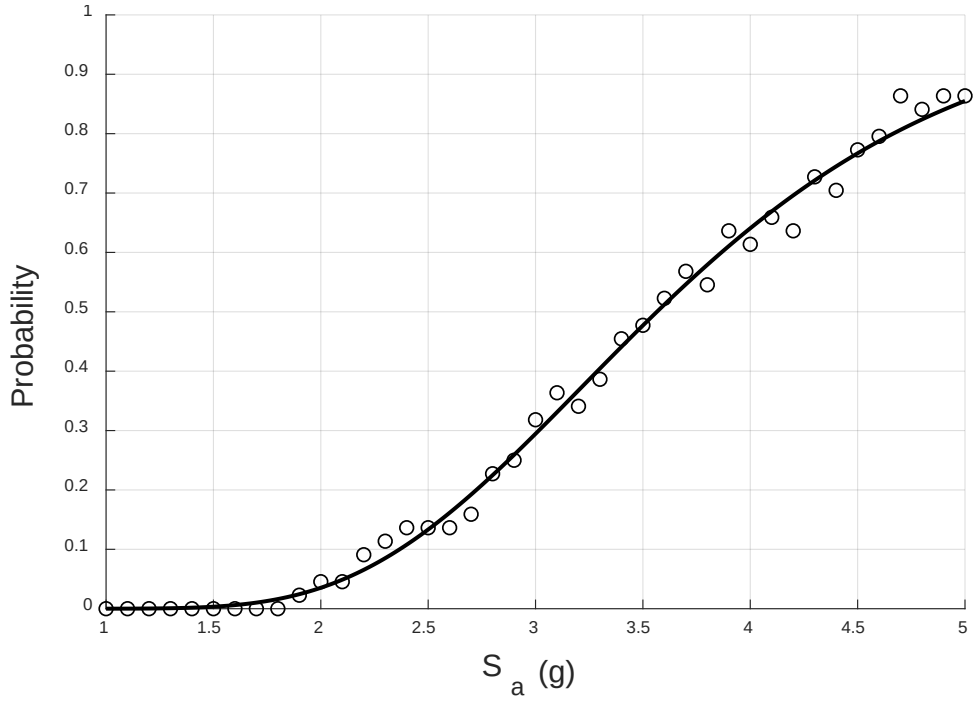


Figure I172. Archetype 100 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

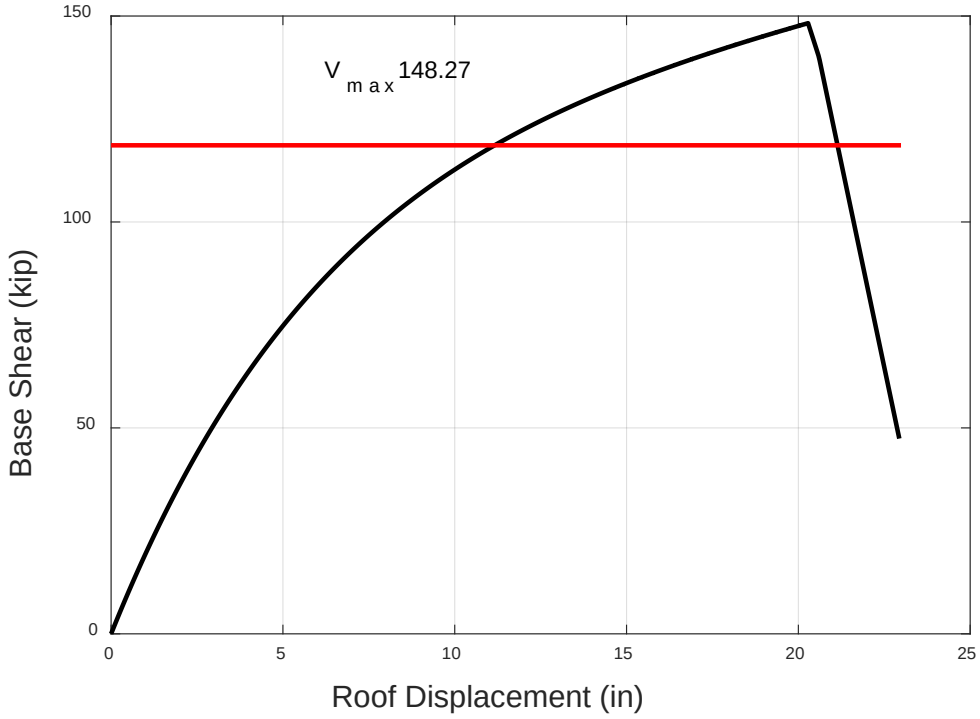


Figure I173. Archetype 101, R = 4, static pushover.

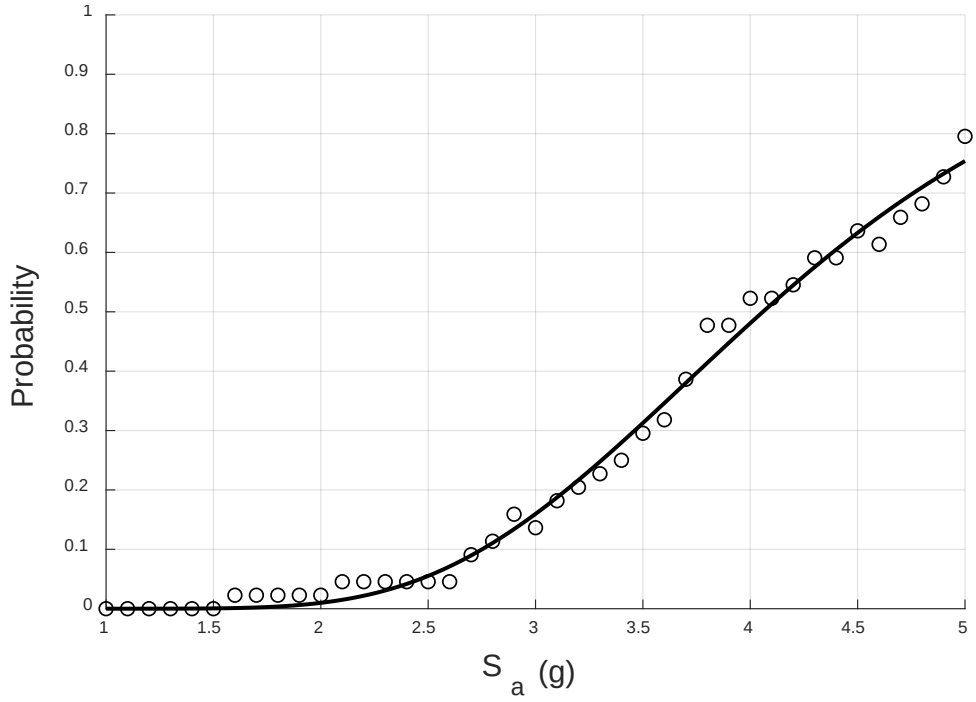


Figure I174. Archetype 101 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

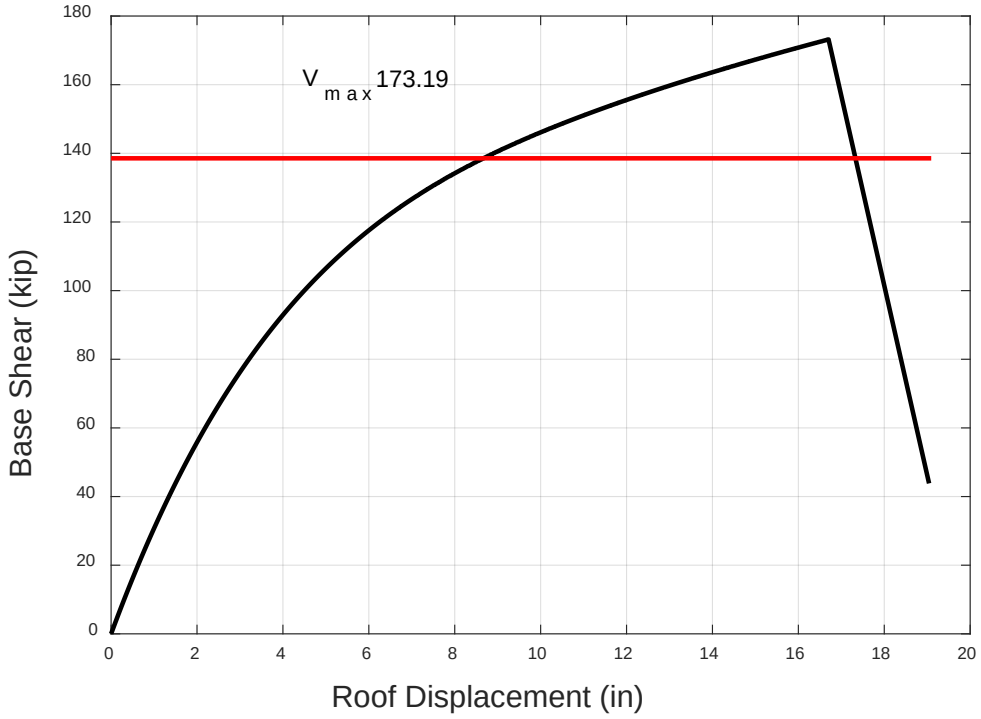


Figure I175. Archetype 102, R = 4, static pushover.

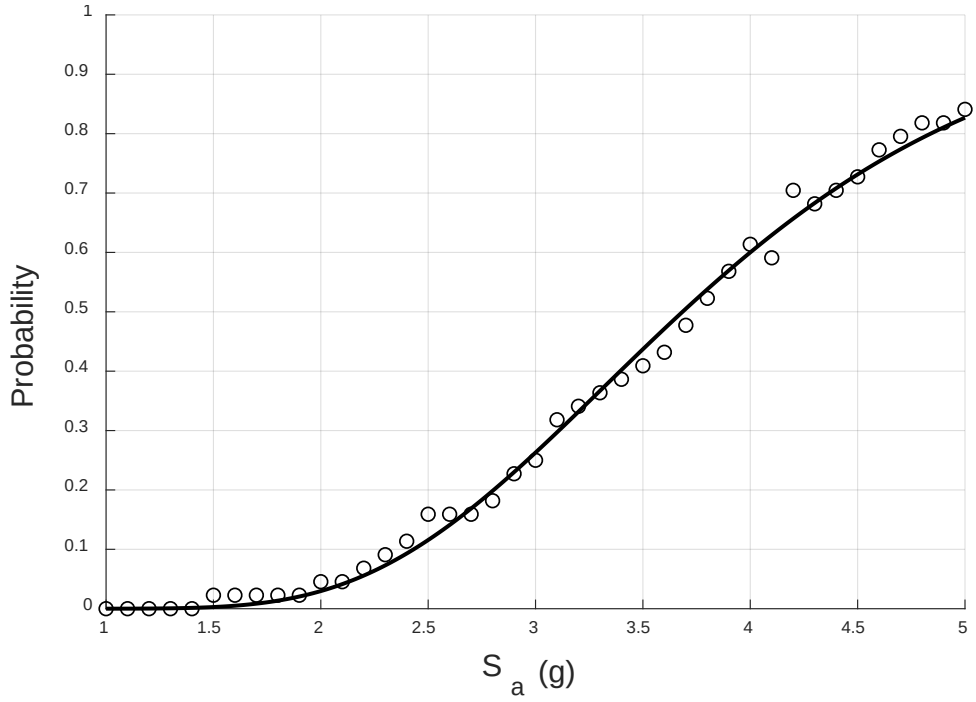


Figure I176. Archetype 102 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

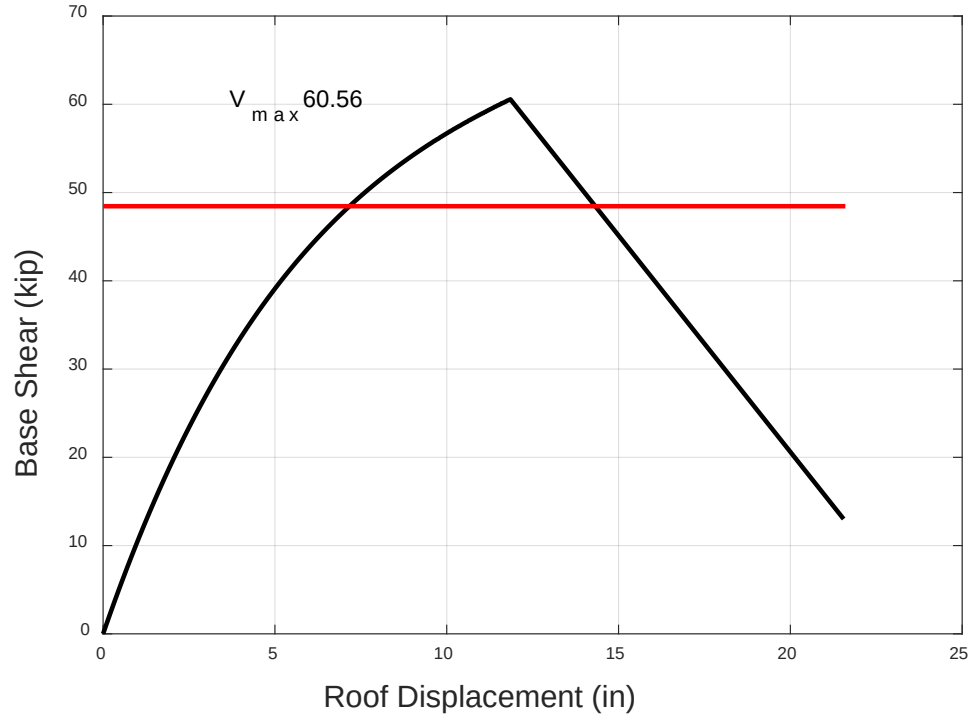


Figure I177. Archetype 109, R = 4, static pushover.

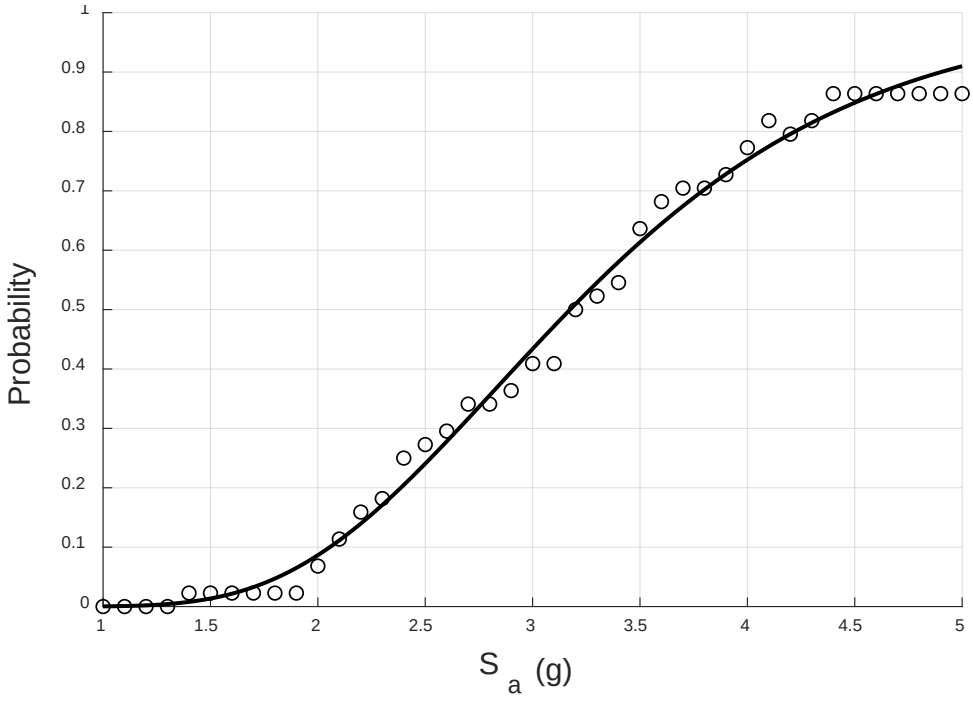


Figure I178. Archetype 109 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

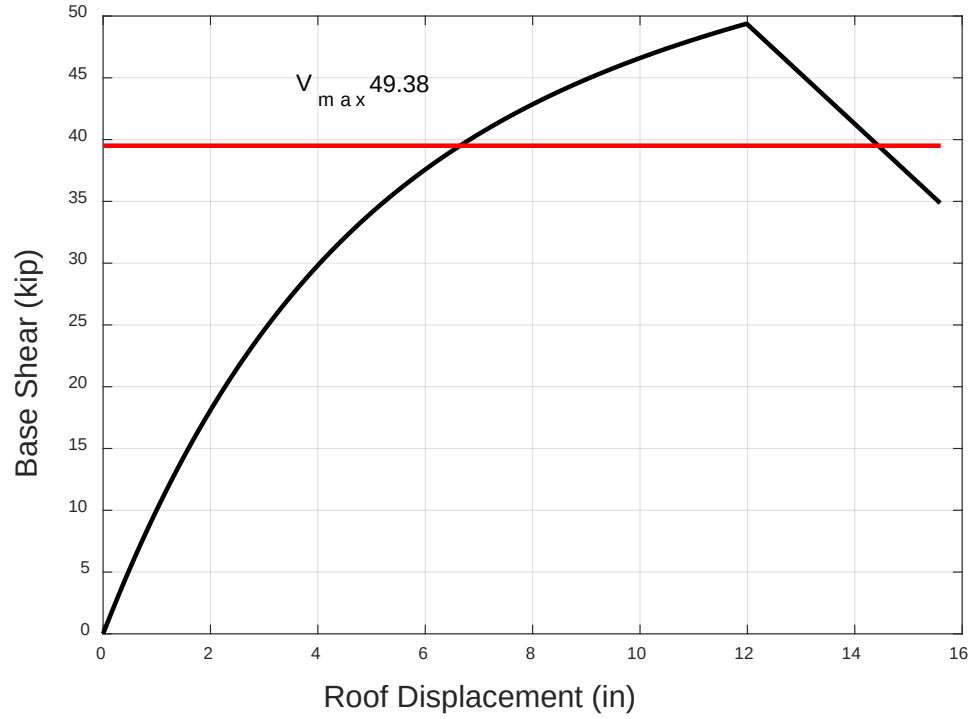


Figure I179. Archetype 110, R = 4, static pushover.

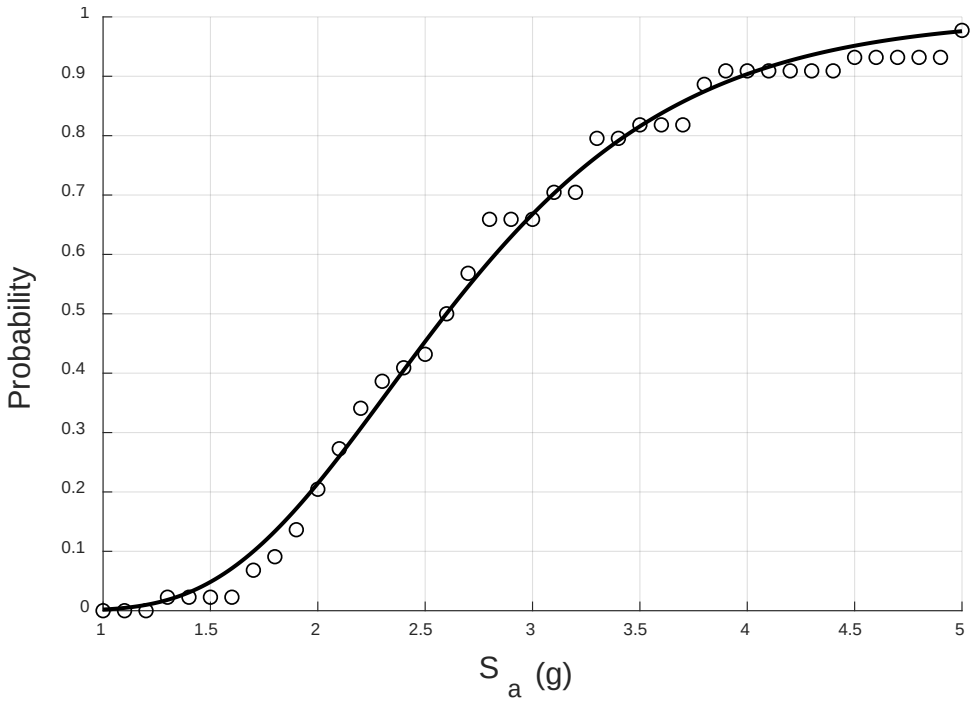


Figure I180. Archetype 110 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

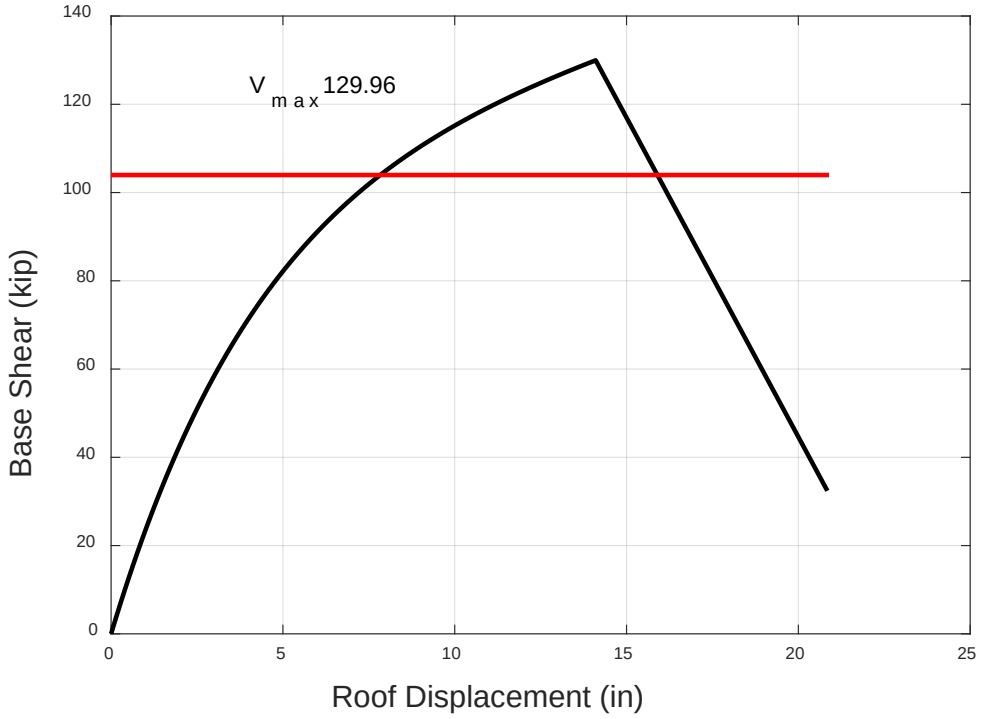


Figure I181. Archetype 111, R = 4, static pushover.

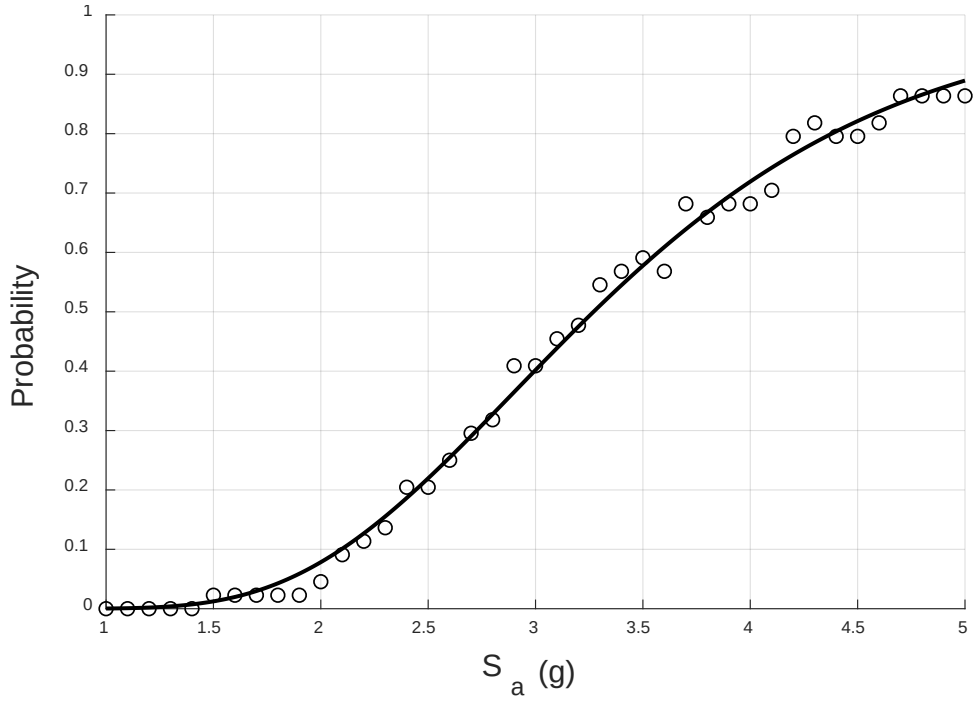


Figure I182. Archetype 111 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

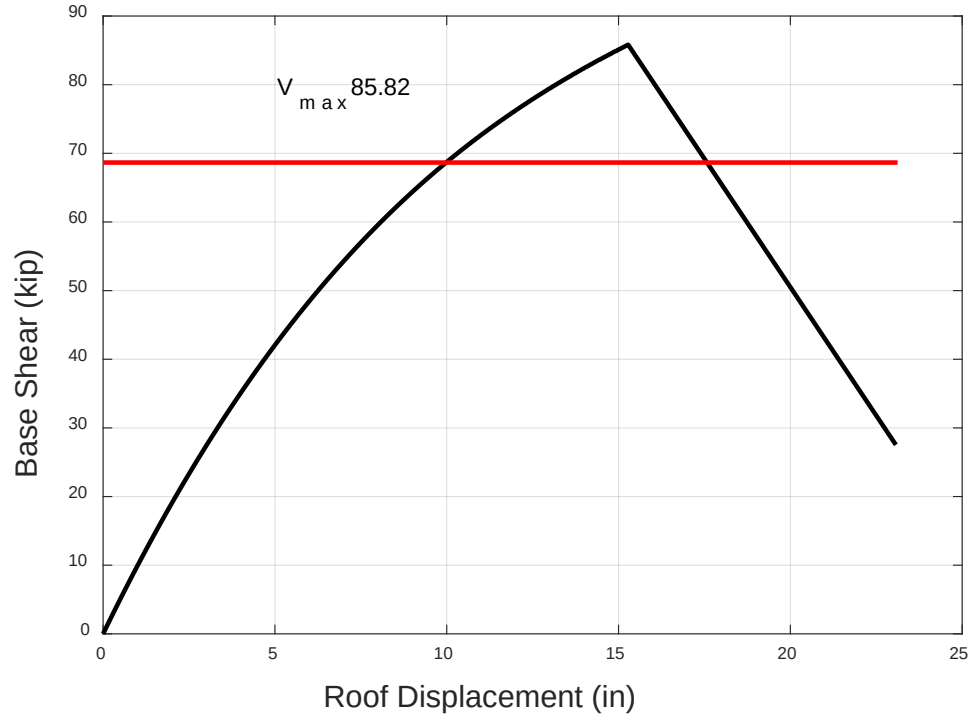


Figure I183. Archetype 112, R = 4, static pushover.

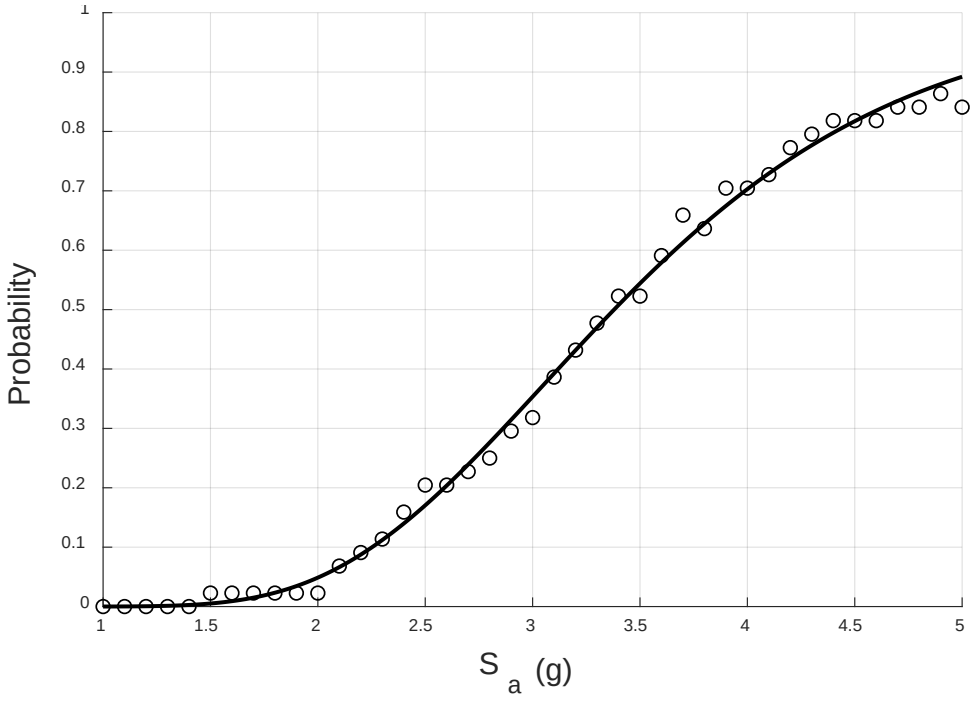


Figure I184. Archetype 112 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

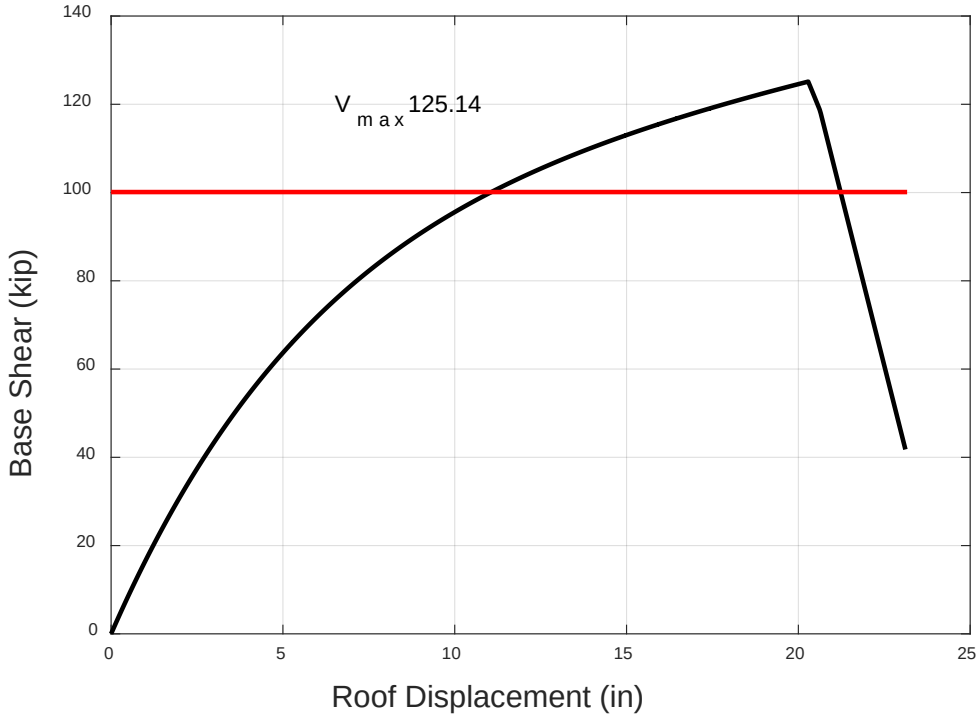


Figure I185. Archetype 113, R = 4, static pushover.

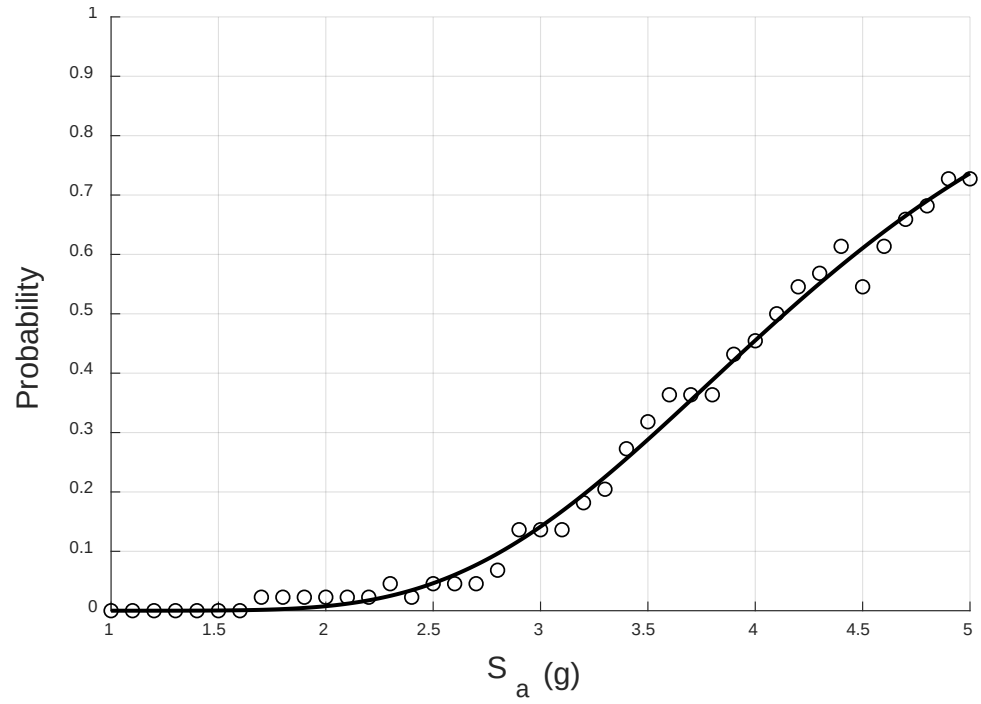


Figure I186. Archetype 113 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

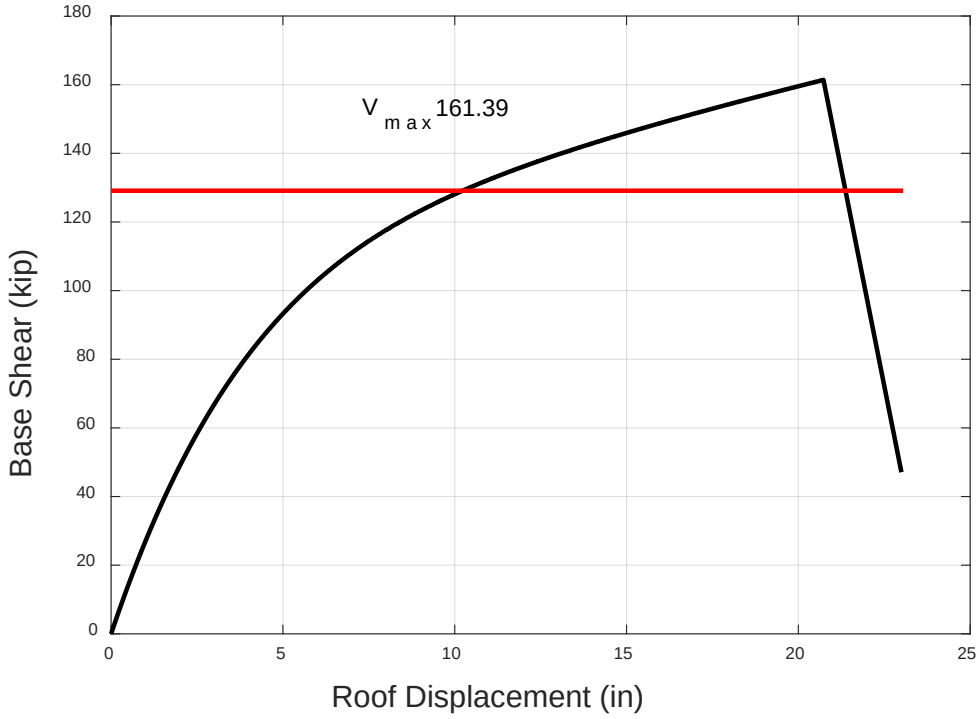


Figure I187. Archetype 114, R = 4, static pushover.

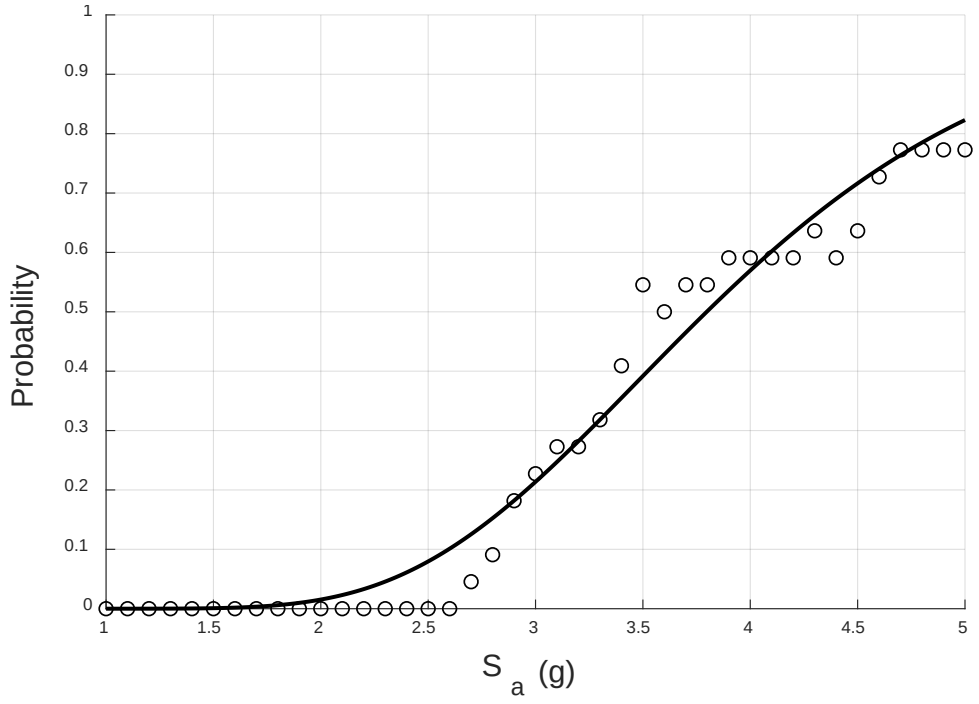


Figure I188. Archetype 114 collapse fragility, R = 4, 2% damping, 5.5% interstory drift limit.

Appendix J: Archetype Deformation Shapes

Deformation shapes for select archetypes are provided in Figures J1 through J18. Details for these archetypes, which ranged from 1 to 6 stories, are shown in Table J1. Incremental dynamic analyses (IDAs) were performed to establish the story locations at which failure occurred for all archetypes subjected to all ground motions. The plots provided were captured as points-in-time when any story reached the nonsimulated collapse interstory drift equal to 4.5% for low and mixed aspect ratio cases and 5.5% for the high aspect ratio case. Plots are not maxima for each story for a particular ground motion. IDAs were performed with increasing spectral acceleration (S_a) ranging from 1.0 *g* to 5.0 *g* in 0.1-*g* increments, resulting in 41 deformation plots for each archetype. For Figures J1 to J18, S_a of 3.5 *g* was chosen to present sample plots with high occurrence of failure. In developing the plots, the maximum interstory drift was limited to 8.4 in. (7% interstory drift), just for the plotting purposes, and the red dots indicate stories associated with failure. Each line in the plot indicates a ground motion; therefore, each plot has 44 lines. In some plots, the appearance of fewer than 44 lines is because certain lines are overlaid.

In general, nonsimulated collapse occurred without a clear pattern from archetype to archetype, occurring at different stories, and was felt to demonstrate that the design method was serving its purposes and not producing a concentration of failures in a particular story. Some failures occurred in a first-mode response, but for the archetypes with more stories, this was not always the case; failure occurred at different stories including the upper stories of the building, depending on the ground motion and/or archetype.

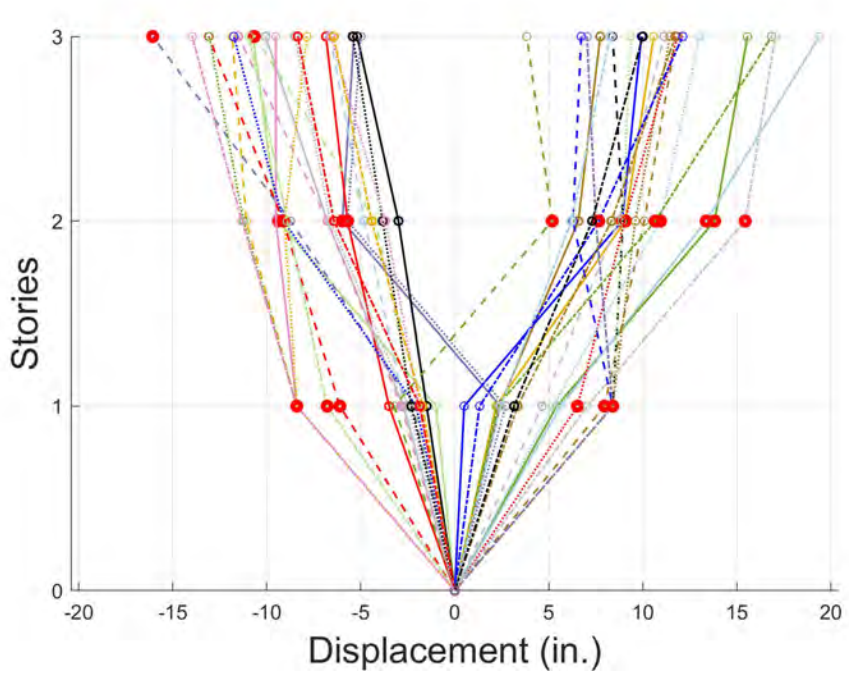


Figure J1. Archetype 01 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 *g*.

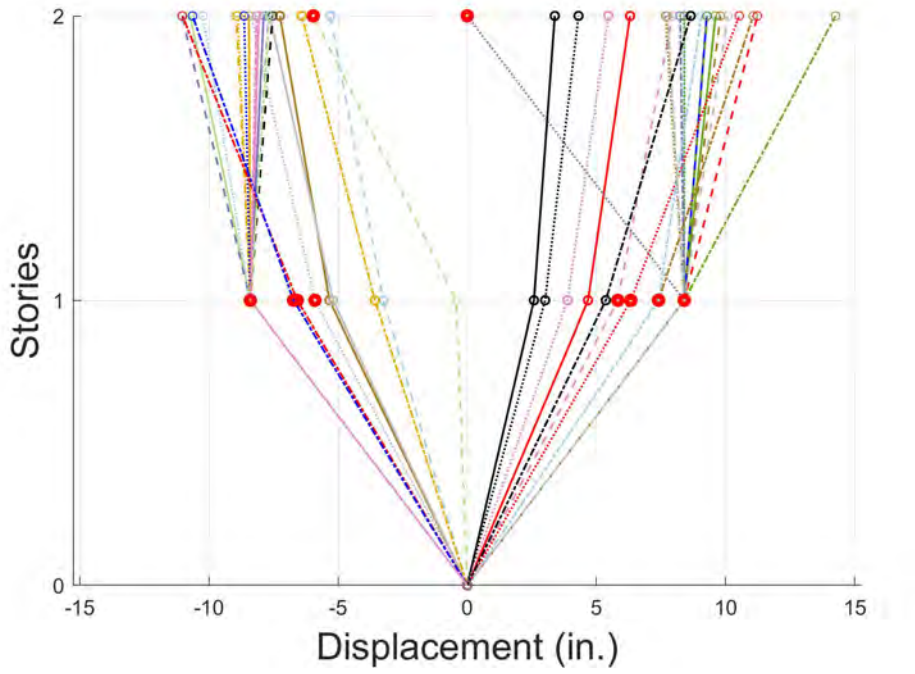


Figure J2. Archetype 02 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 *g*.

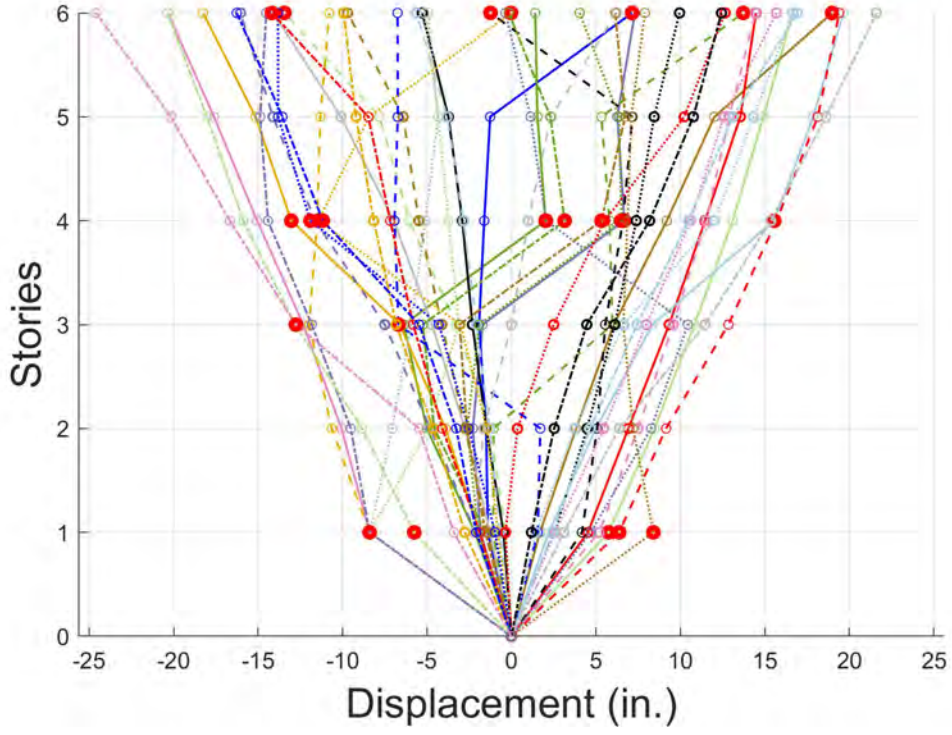


Figure J3. Archetype 04 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

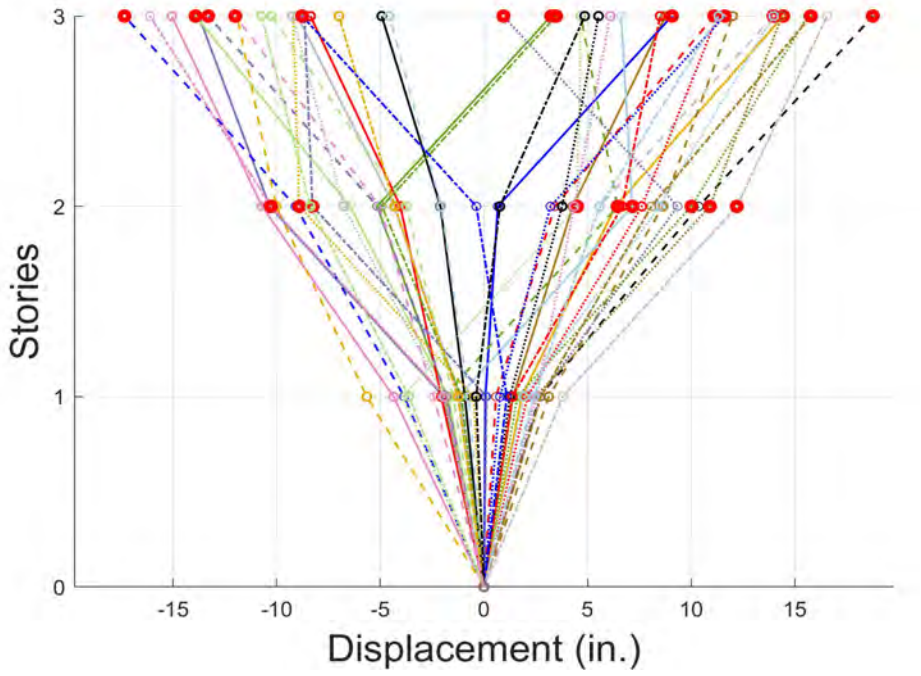


Figure J4. Archetype 25 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

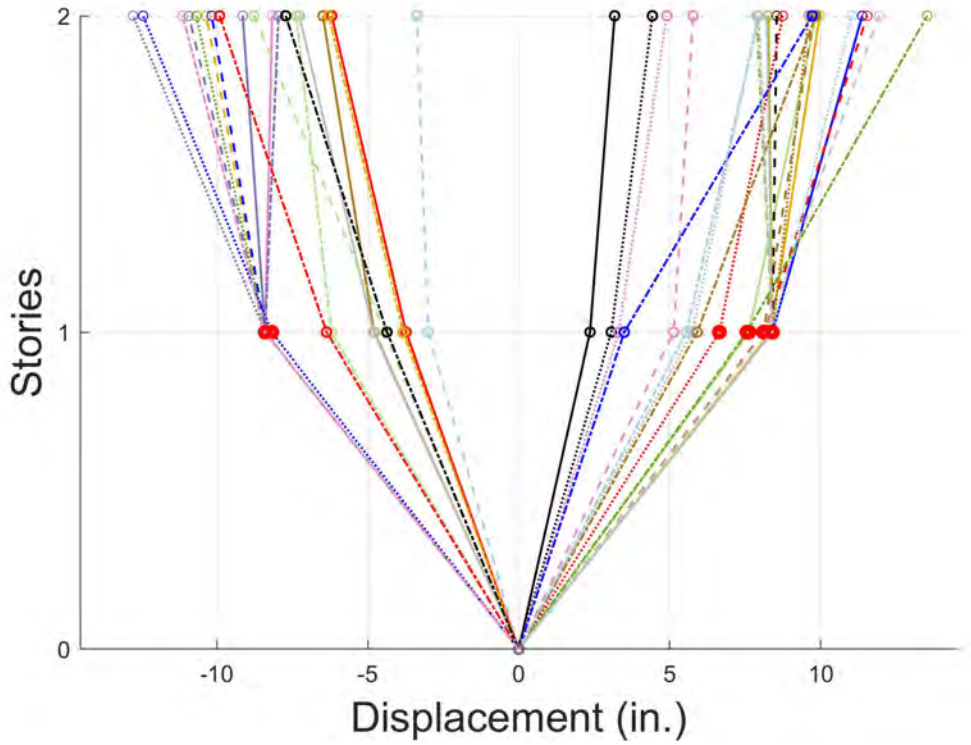


Figure J5. Archetype 26 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

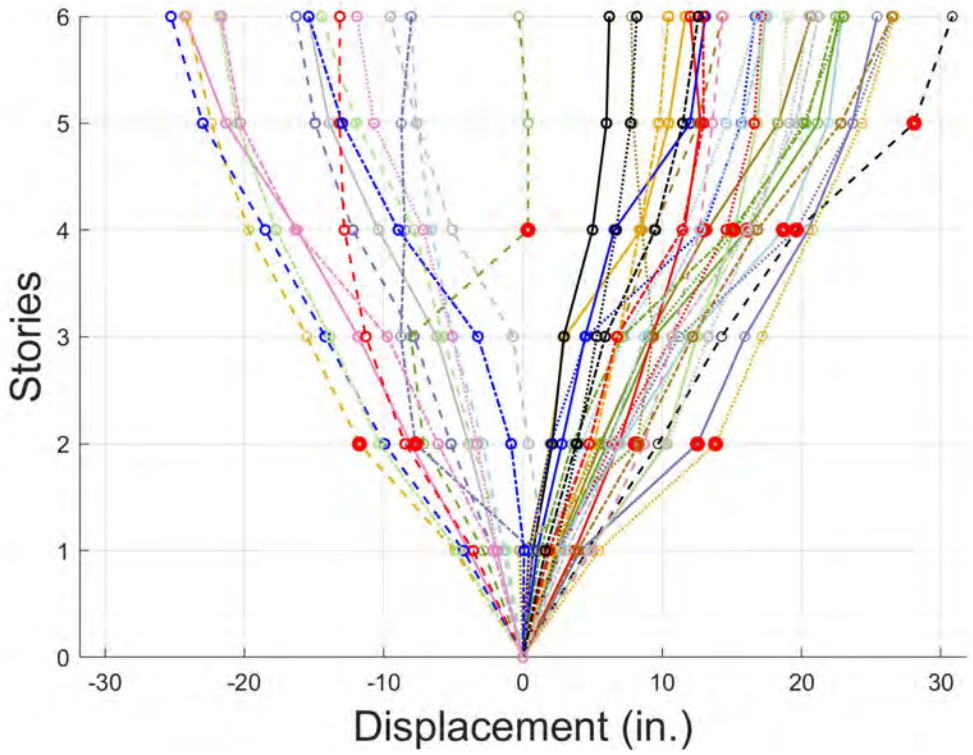


Figure J6. Archetype 28 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

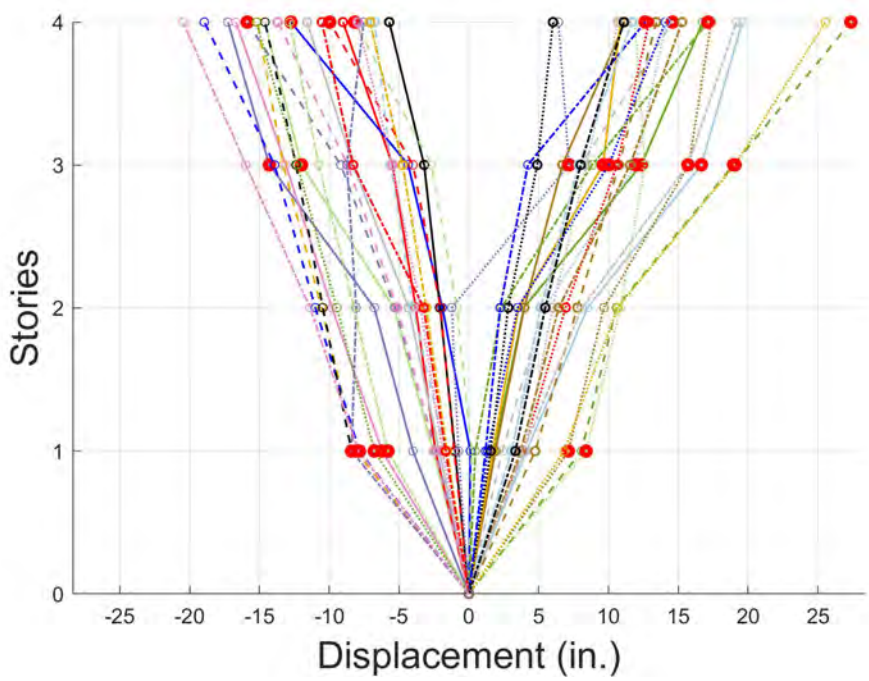


Figure J7. Archetype 49 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

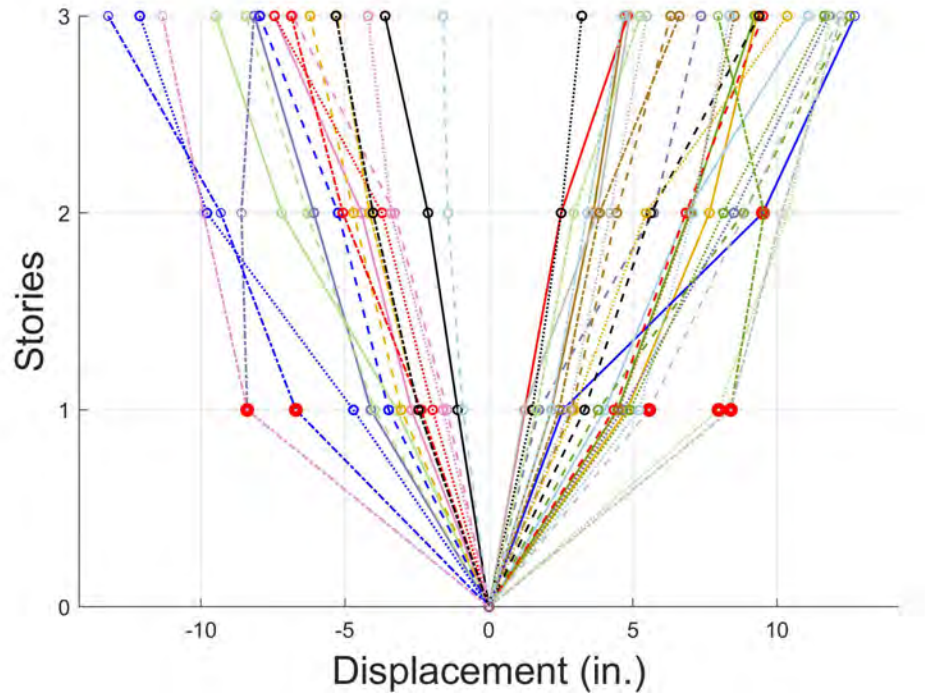


Figure J8. Archetype 51 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

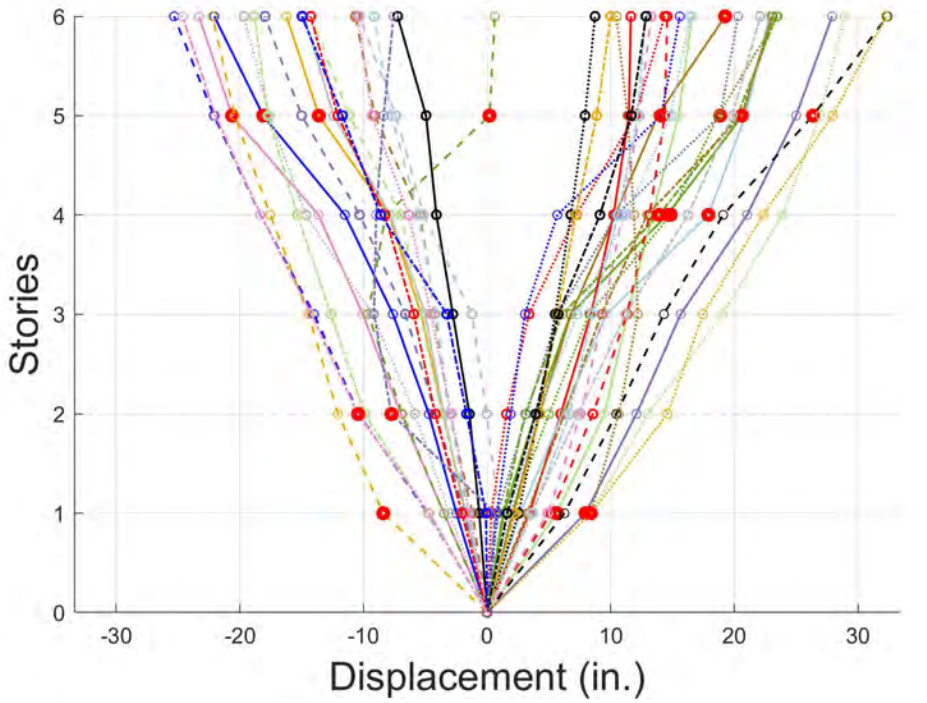


Figure J9. Archetype 52 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

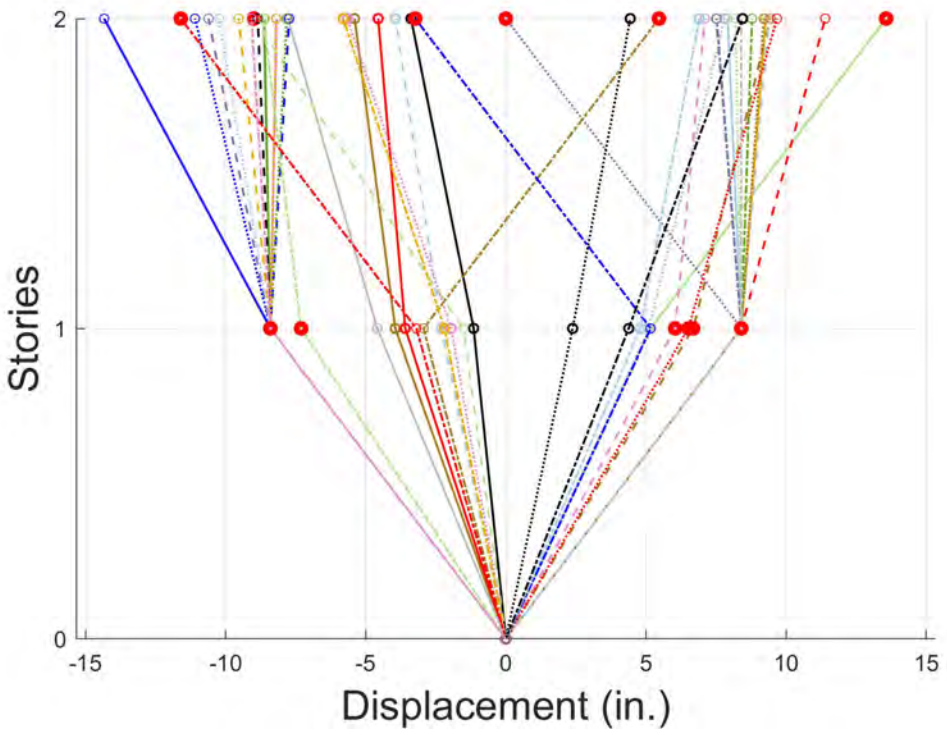


Figure J10. Archetype 73 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

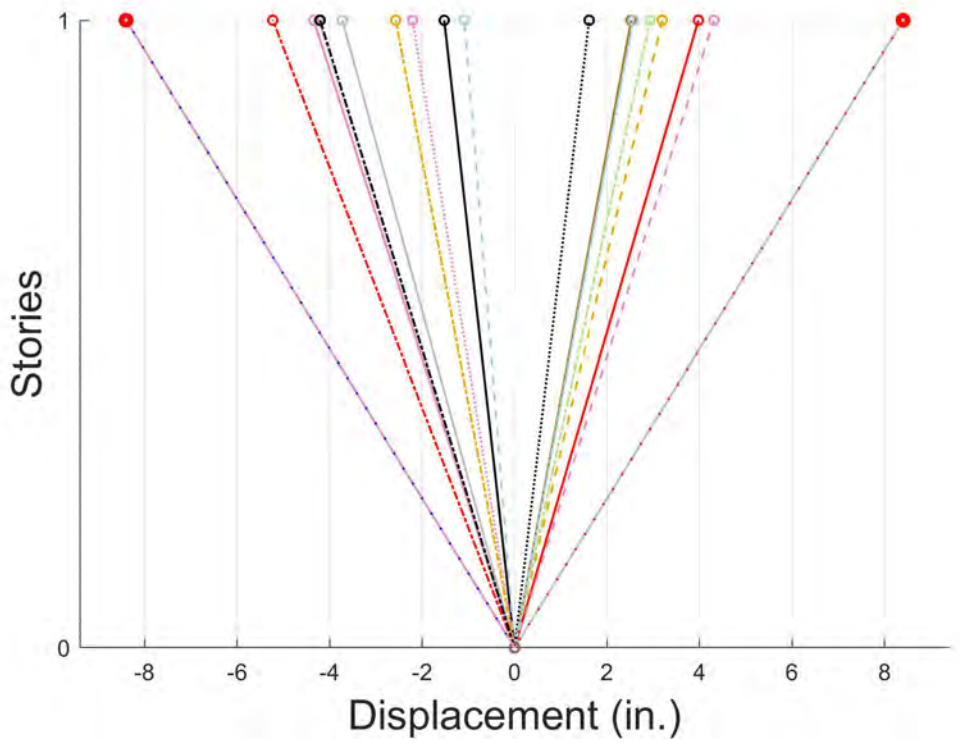


Figure J11. Archetype 74 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

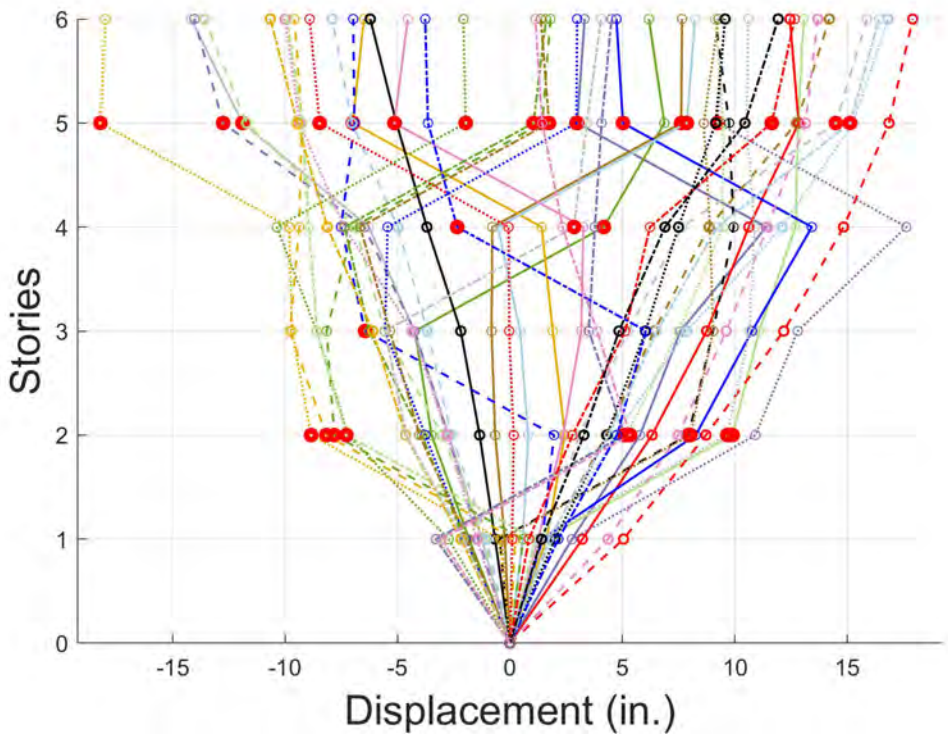


Figure J12. Archetype 76 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

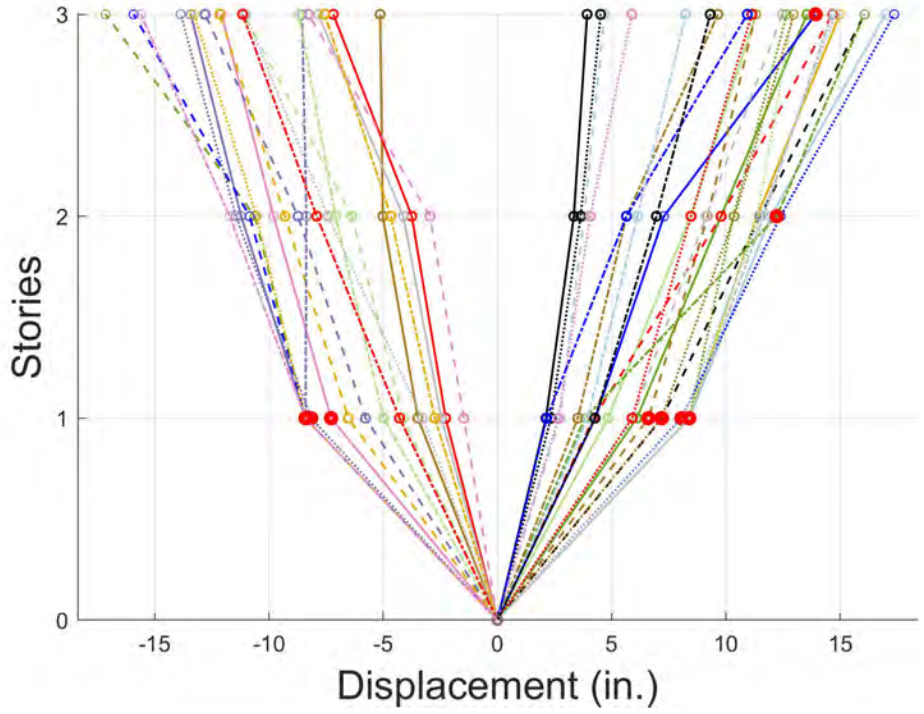


Figure J13. Archetype 97 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

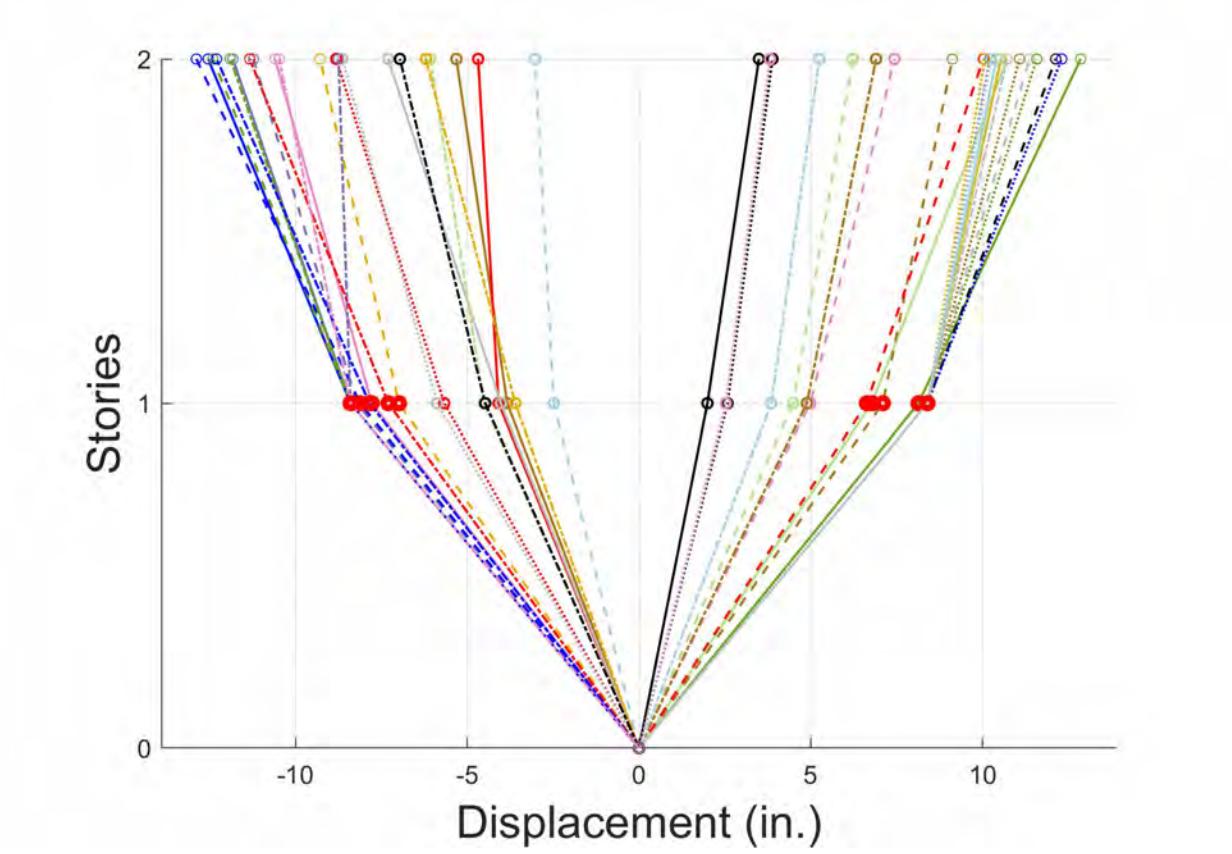


Figure J14. Archetype 98 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

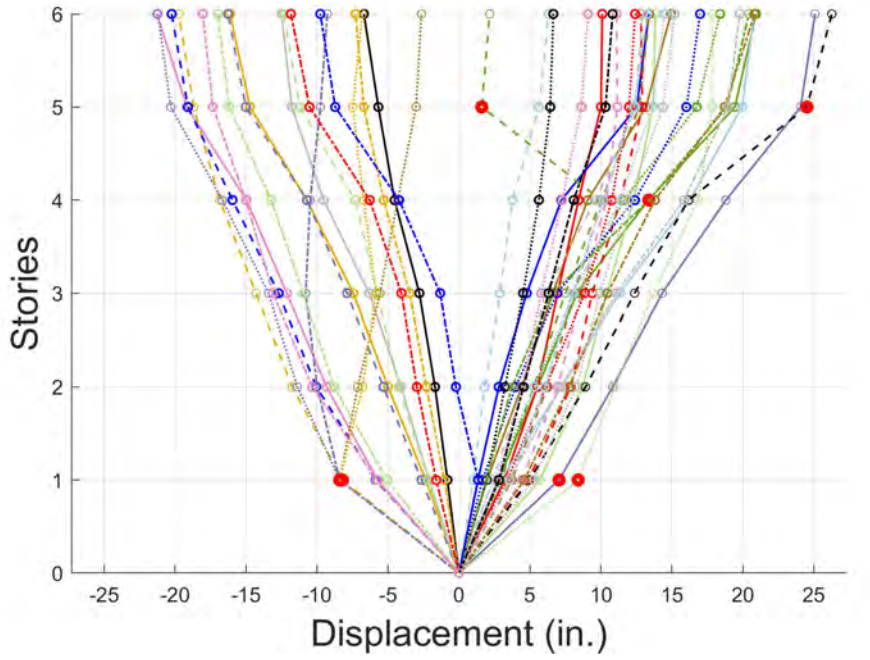


Figure J15. Archetype 101 collapse fragility, R = 3, 2% damping, 5.5% interstory drift limit, S_a = 3.5 g.

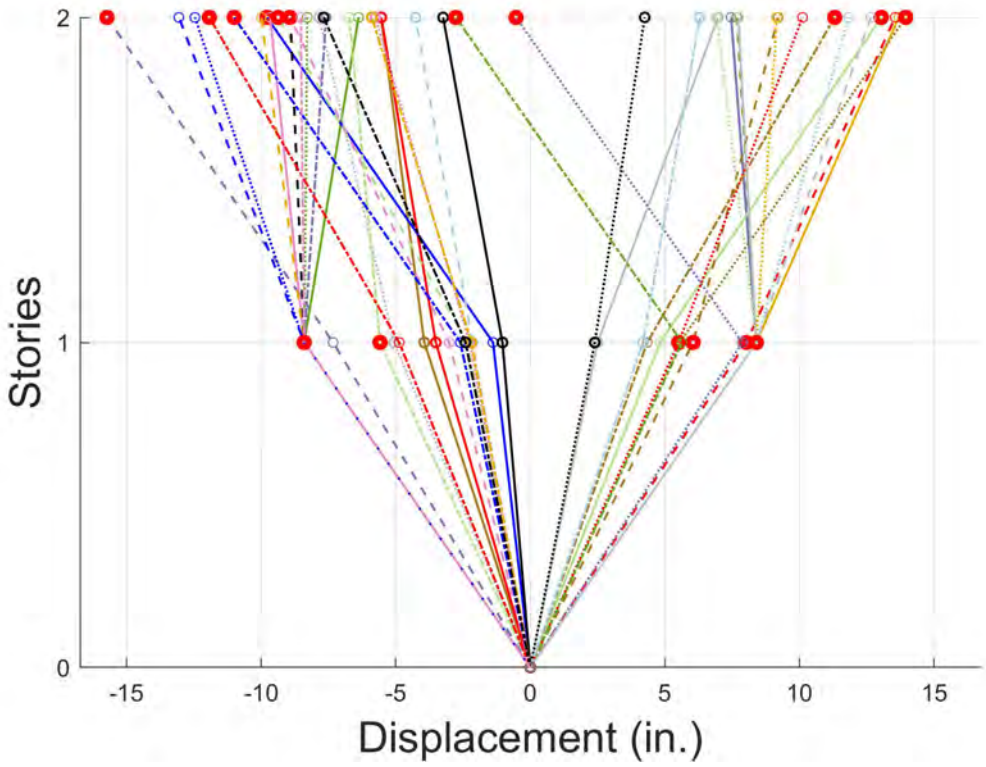


Figure J16. Archetype 121 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

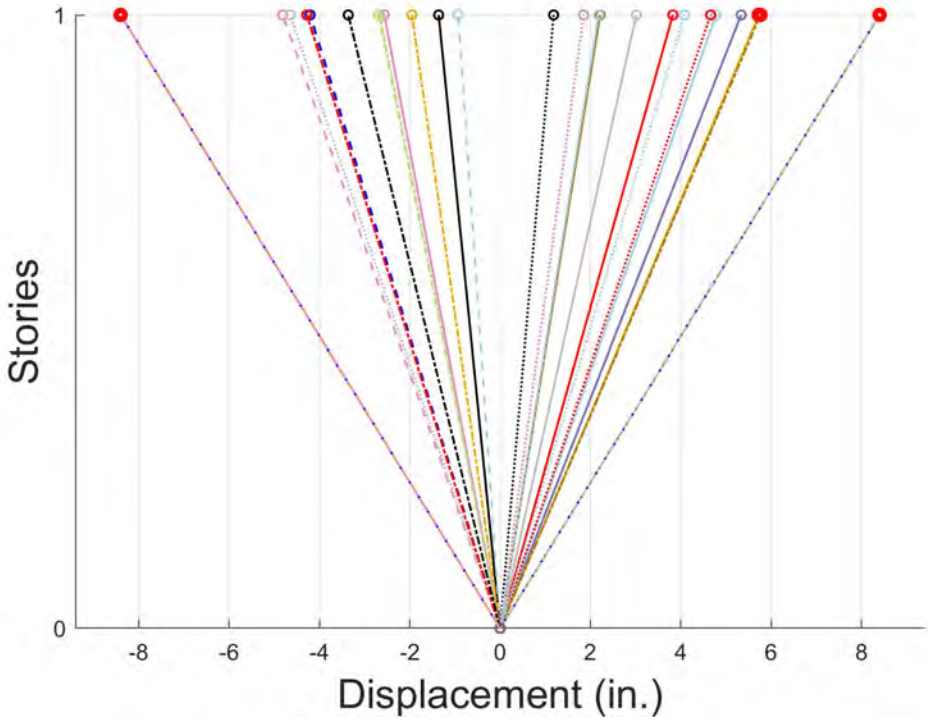


Figure J17. Archetype 122 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

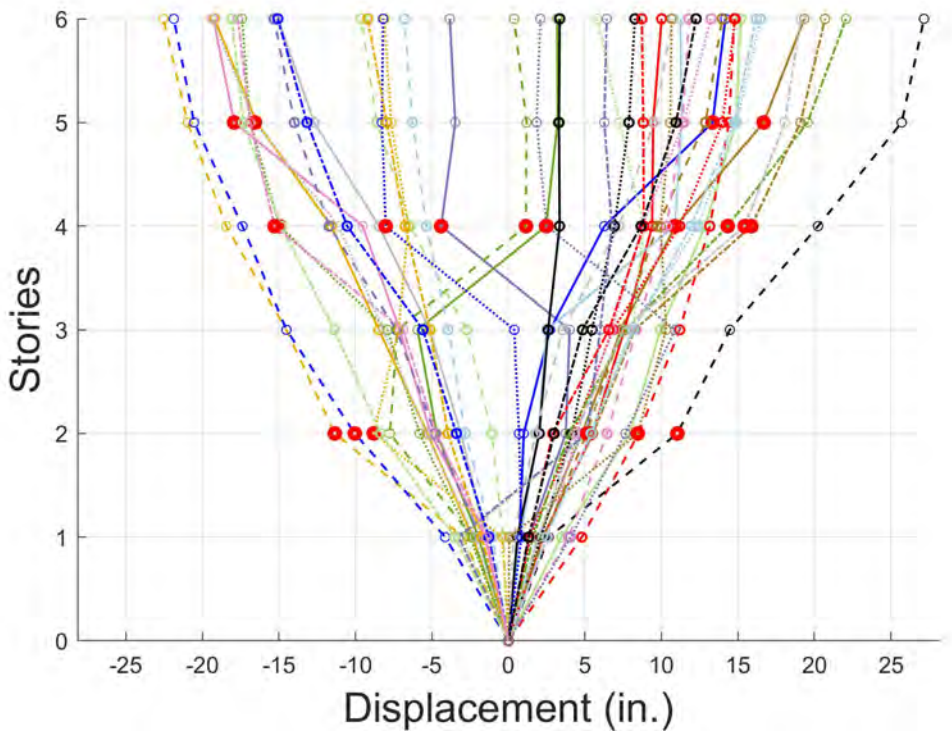


Figure J18. Archetype 124 collapse fragility, R = 3, 2% damping, 4.5% interstory drift limit, S_a = 3.5 g.

Table J1—Select archetypes for demonstrating deformation shapes

Group No.		Grouping criteria: Basic config.	Design load level: Gravity	Design load level: Seismic	Grouping criteria: Period domain	Archetype description	Archetype No.
PG-1	2.5–20 ft wall	Low aspect ratio panels	High	SDC D_{max}	Short	1_E_3_1_LR_HG_DX_SP	1
						2_4_2_1_LR_HG_DX_SP	2
PG-2					Long	4_3_ 6_1_LR_HG_ DX_LP	4
PG-9		High aspect ratio panels	High	SDC D_{max}	Short	1_E_3_1_HR_HG_DX_SP	25
						2_4_2_1_HR_HG_DX_SP	26
PG-10					Long	4_3_ 6_1_HR_HG_ DX_LP	28
PG-17		Mixed aspect ratio	High	SDC D_{max}	Short	8_3_4_1_MR_HG_DX_SP	49
						7_3_3_1_MR_HG_DX_SP	51
PG-18					Long	4_3_ 6_1_MR_HG_ DX_LP	52
PG-25	20–60 ft wall	Low aspect ratio panels	High	SDC D_{max}	Short	2_2_2_2_ LR_HG_ DX_SP	73
						3_2_1_2_ LR_HG_ DX_SP	74
PG-26					Long	4_E_6_2_ LR_HG_ DX_LP	76
PG-33		High aspect ratio panels	High	SDC D_{max}	Short	1_A_3_2_ HR_HG_ DX_SP	97
						6_E_2_2_ HR_HG_ DX_SP	98
PG-34					Long	8_B_6_2_ HR_HG_ DX_LP	101
PG-41		Mixed aspect ratio	High	SDC D_{max}	Short	2_2_2_2_ MR_HG_ DX_SP	121
						3_2_1_2_ MR_HG_ DX_SP	122
PG-42					Long	4_E_6_2_ MR_HG_ DX_LP	124