DISSERTATION

DETERMINATION OF SEISMIC PERFORMANCE FACTORS FOR CROSS LAMINATED TIMBER SHEAR WALL SYSTEM BASED ON FEMA P695 METHODOLOGY

Submitted by

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In partial fulfillment of the requirements

For the Degree of Doctor of Philosophy

Colorado State University

Fort Collins, Colorado

Fall 2018

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ABSTRACT

DETERMINATION OF SEISMIC PERFORMANCE FACTORS FOR CROSS LAMINATED TIMBER SHEAR WALL SYSTEM BASED ON FEMA P695 METHODOLOGY

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 utilized 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 US.

The purpose of this study was to investigate the seismic behavior of CLT based shear wall systems and 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 that include, single-family dwellings, multi-family dwellings, and commercial (including mixed-use) mid-rise buildings were developed. Archetypes were then extracted from these index buildings. Testing performed at the component and subassembly levels include connector tests and isolated shear wall tests. A subsequent full-scale shake table

test was performed for system level demonstration. A critical aspect of this study is use of generic connectors whose properties are already addressed by a design specification to facilitate building 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.

Connector tests were performed on angle brackets, used for attachment of the wall to the supporting element, and inter-panel connectors. These tests showed connector thickness to be important in achieving the desired ductile behavior with lesser thickness (12 gauge) being the more favorable. Quasi-static 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 the wall performance, i.e. higher strength and deformation capacity. Specific gravity also had a significant effect on wall behavior while CLT thickness was less influential. Higher aspect ratio panels (4:1) demonstrated lower stiffness and substantially larger deformation capacity compared to moderate aspect ratio panels (2:1). However, based on the test results there is likely a lower bound for aspect ratio (at 2:1) where it ceases to benefit deformation capacity of the wall. Multi-panel configuration comprised of high aspect ratio panels connected through vertical joint demonstrated considerably larger deformation capacity. Shake table tests showed the proposed system's potential to meet life-safety code requirements and its applicability in US seismic regions.

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, an

R factor of 3 is proposed for the CLT shear wall systems and an R factor of 4 is proposed for the cases with high aspect ratio panels only. Results from the study will be proposed for implementation in the seismic design codes and standards in the US.

ACKNOWLEDGEMENTS

I would like express my sincerest gratitude to my advisor, Dr. van de Lindt, for providing me guidance, support and encouragement throughout this research and my entire graduate study. His expertise and leadership enabled me to develop an understanding of the subject and were crucial to the successful fulfillment of this study. He has been an incredible mentor and a great source of inspiration. I'll always be grateful to him.

I am especially indebted to Mr. Philip Line (American Wood Council) for being a constant resource and providing invaluable expertise. He had considerable contributions in various aspects of this project and his help is greatly appreciated. I also wish to thank Mr. Doug Rammer (USDA Forest Products Laboratory) for providing us with the resources to complete this study. This study would not have been possible without his help.

I would like to extend my gratitude to my committee members, Dr. Husaam Mahmood, Dr. Paul Heyliger and Dr. Bolivar Senior; my project collaborators, Dr. Shiling Pei (Colorado School of Mines) and Dr. Marjan Popovski (FPInnovations); and the project peer panel, Dr. Charlie Kircher (Charles Kircher & Associates), Dr. Dan Dolan (Washington State University) and Ms. Kelly Cobeen (WSJ Associates Inc.) for valuable advice and suggestions throughout this study. I acquired invaluable knowledge through our interactions for which I am grateful.

This study was funded by a joint venture agreement between the United States Department of Agriculture Forest Products Laboratory (FPL) and Colorado State University - USDA-USFS, 16-JV-11111133-036. That support is gratefully acknowledged. Structurlam Products LP and Nordic Structures both provided partial donation of CLT for this project and their contributions are appreciated.

Mr. Junior Garza and Mr. Matthew Szydlowski, staff members of the Structures Laboratory at Colorado State University, are acknowledged for their assistance with the testing part of this project. Many thanks to my friends and colleagues for their support and encouragement. I cannot leave CSU without mentioning Ms. Christy Eylar, who provided help and guidance on various matters. Last but not least, I would like to thank my parents and my sisters for their love and support and I am very grateful for all the sacrifices my parents made to make this educational journey possible.

DISCLAIMER

This study is the result of a joint venture agreement between the United States Department of Agriculture Forest Products Laboratory (FPL) and Colorado State University - USDA-USFS, 16-JV-11111133-036. Until such time as the final report is published, this dissertation should be regarded as information only.

DEDICATION

To my beloved parents

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CHAPTER 1: INTRODUCTION

1.1 Background

Wood is a versatile construction material that has been used over 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 multi-family housing, commercial buildings, and bridges. With the recent shift towards sustainability and the concept of 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 makes wood a preferred green building material (FPL, 2010; Ritter et al., 2011).

Wood is an orthotropic material meaning that it has unique properties in mutually perpendicular directions that are longitudinal, radial, and tangential. While variation in properties is common in all material, wood properties vary considerably since it is a natural material. As a result Engineered Wood Products (EWP) or wood-based composites were introduced in order to ensure uniformity and control over the range of properties, to efficiently utilize 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 in. to 2 in. and the width varies from 2.4 in. to 9.5 in. (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 to 2005, Cross Laminated Timber (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 termed X-Lam offers a number of advantages; mass production, prefabrication, speed of construction, 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).

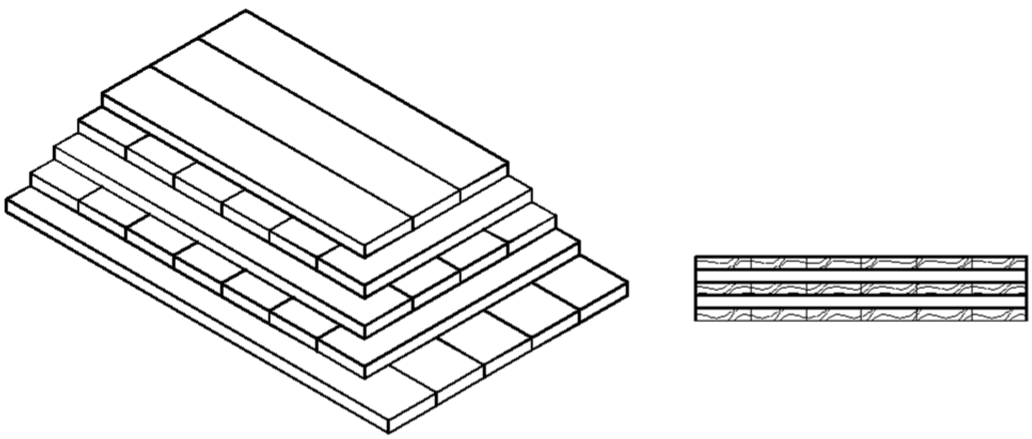


Figure 1.1: CLT Panel Configuration

Applications of CLT vary widely and include residential buildings, industrial and commercial buildings, and bridges. However, it is the multi-story 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 mid-rise construction. Numerous CLT buildings have been erected around the world in Europe, Australia, and recently in North America. Table 1.1 provides a list of these major CLT structures and the 9 story Stadthaus is shown in Figure 1.2.

Table 1.1. CLT multistory structures[a]

Project	**Location**	**Height (story)**	**Comments**
Stadthaus, Murray grove[b]	London, UK	9	• Timber erected in just nine 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, 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[c]	Milan, Italy	9	• Four 9-story buildings • Project is a result of competition to provide an innovative approach to social housing experiment

[a] Forestry Innovation Investment and Binational Softwood Lumber Council (2014)
[b] Yate et al. (2008)
[c] KLH
[d] Bernasconi (2016)

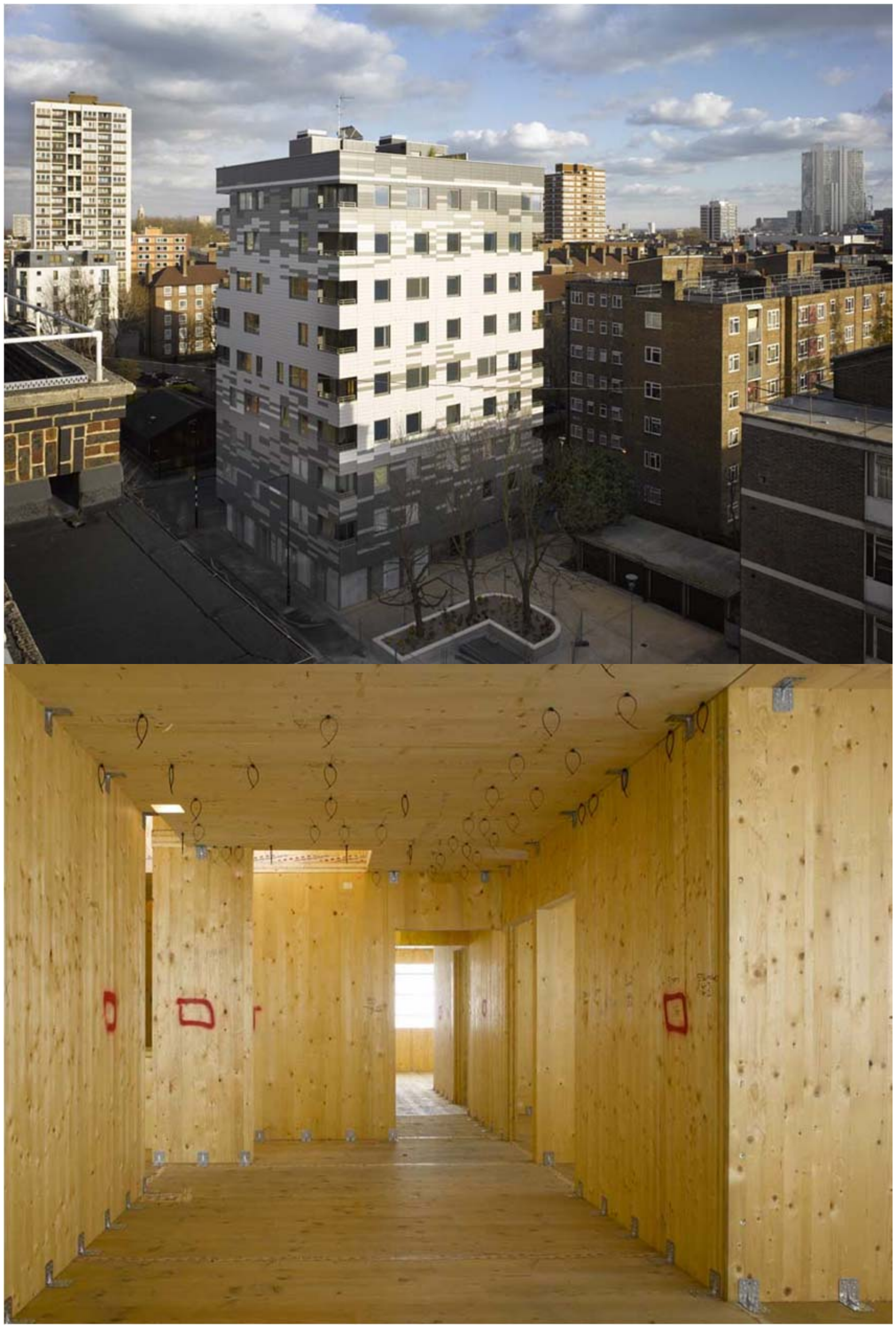


Figure 1.2: Stadthaus photos, Photographer Will Pryce

1.2 Motivation and Need

Despite many advantages, a 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 mid-rise 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).

Mohammad et al. (2012) identified a multi-level strategy that includes development of a product standard, material design standard, and their subsequent adoption into the building codes. In the US, there has been recent development on all these fronts that included publication of ANSI/APA PRG320, the North American Standard for Performance-Rated Cross-Laminated Timber (2012), addition of a chapter dedicated to CLT in the 2015 edition of the National Design Specification® for Wood Construction (NDS®) (ANSI/AWC, 2015) and recognition of CLT in the 2015 International Building Code (IBC, 2015).

One area that requires attention is the development of seismic performance and behavior of CLT lateral systems. In the early stages of its development in Europe, CLT structures were mainly constructed in low seismic regions. Although CLT was introduced over two decades ago, it was in the past decade that researchers started focusing on utilizing CLT as a lateral force resisting system and this triggered an increase in the number of studies geared towards 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 utilized as a lateral force resisting system. With the introduction of CLT to the US construction market and the current modern urbanization trend (Alig et al., 2004), many believe that it can fill a gap for certain regions of the US, specifically, the mid-rise condominium, commercial, and mixed-use building market in seismic regions.

CLT based Seismic Force Resisting Systems (SFRS) are not recognized in current US 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 against 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 FEMA P695 (FEMA, 2009) methodology which is a systematic approach that integrates the design method, experimental results, nonlinear static and dynamic analyses, incorporates uncertainties, and includes peer oversight. An overview of the methodology is presented in the next section.

One important aspect of this study was the use of non-proprietary components and connectors already addressed by US design codes to facilitate building 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 dissertation research are as follows:

- 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, subassembly, and system 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 Overview of the FEMA P695 Methodology

As mentioned earlier, the FEMA P695 methodology is used to evaluate seismic performance factors known as the response modification factors, R, overstrength factor, Ω_o and deflection amplification factor, C_d. R is defined as the ratio of the shear developed in the system if the system were to remain entirely linearly elastic under design ground motions V_E to the design base shear value V. Ω_o is the ratio of maximum shear strength V_{max} of the yielded system to the design base shear. C_d is defined as the ratio of the roof drift of the yielded system under design earthquake ground motions δ to the roof drift under design base shear considering the system to behave linearly elastic δ_E, multiplied by the R factor. SPFs are best described using the following equations and illustrated in Figure 1.3.

R = Response Modification Coefficient = V_E/V

Ω_o = Overstrength Factor = V_{max} / V

C_d = Deflection Amplification Factor = $(\delta/\delta_E)\ R$

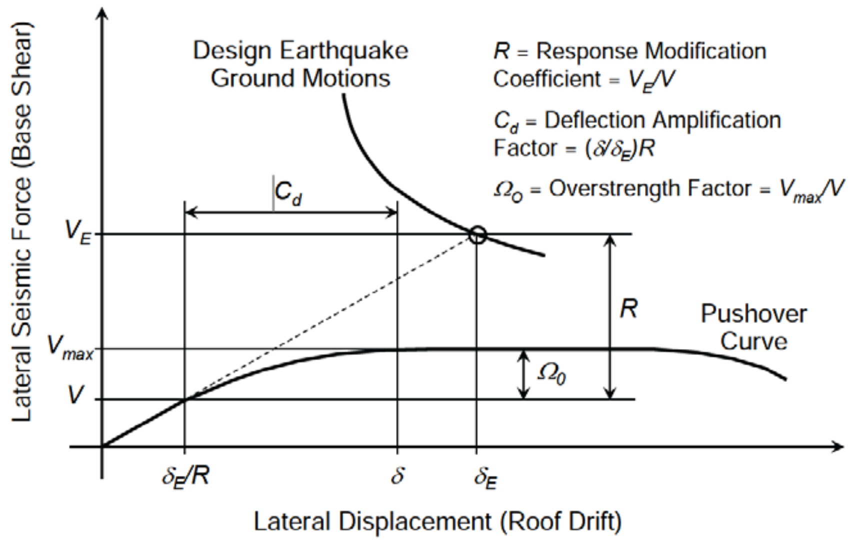


Figure 1.3: Illustration of Seismic Performance Factors (after FEMA P695)

The FEMA P695 procedure is iterative in nature and includes the following steps:

(1) Establish design requirements and develop specifications that are based on applicable codes and standards;

(2) Identification of a number of archetypes to be representative of the full design space from low-rise single family buildings to mid-rise mixed-used buildings including multi-family buildings and office buildings; The archetypes are categorized based on key design variables such as geometric variations, load intensities, and other variables that are known to have an effect on system performance;

(3) A series of experimental tests on panels with varying holddown conditions and aspect ratios; the results are then used to calibrate the nonlinear numerical models;

(4) The development and validation of a nonlinear computer model; the developed model takes into account degradation in stiffness and strength in the inelastic range;

(5) Comprehensive static (pushover) and incremental dynamic analysis (IDA) are performed to compute the median collapse that is then used to evaluate margin against collapse for the archetypes; and

(6) Determination of whether the seismic performance factors are acceptable based on the FEMA P695 requirements.

Figure 1.4 explains the procedure and its flow. An independent team of experts are 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 is to exercise 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, analysis, and final selection of the seismic performance factors. The peer review panel members for this study meet all the criteria specified in the FEMA P695 methodology and are qualified to critically evaluate all aspects of this project. A list of peer review panel members along with their expertise is provided in Table 1.2.

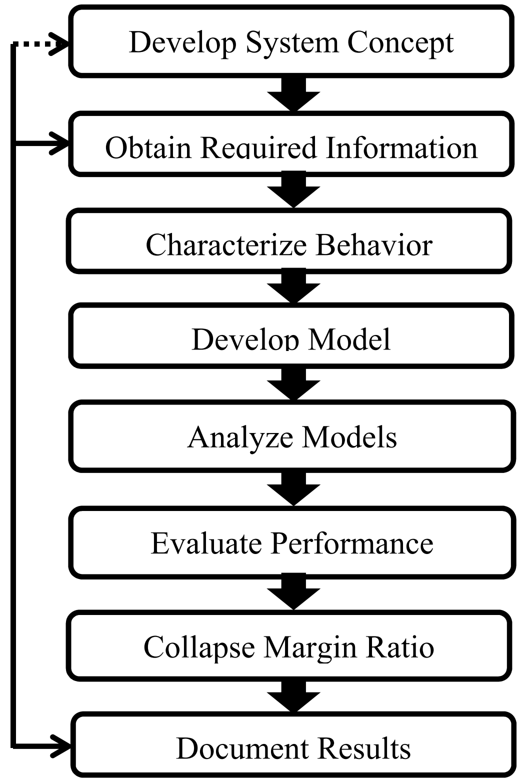


Figure 1.4: Overview of the FEMA P695 methodology

Table 1.2. Peer panel overview

Member	Expertise	Role
Charlie Kircher, Ph.D., P.E. Principal and Owner Charles Kircher & 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.	Structural Wood Design Codes Wood Seismic Design and Detailing Seismic Performance Evaluation Structural Evaluation	Panel Member

1.5 Layout of this Dissertation

Various phases of the project consist of development of the archetypes, design methodology, testing, modeling, and analyses. This dissertation 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 being the birthplace of CLT, most of the seismic studies originated in Europe with two main projects that include 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 in this chapter. A total of 9 index buildings that include single-family, multi-family and commercial configurations are developed and a logical way to extract the 2D archetype from these index buildings is presented.
- Chapter 4: The testing part of this project that includes three phases, namely (i) connector testing (ii) CLT shear wall testing and (iii) shake table testing are presented in this chapter. 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 inter-panel connectors. CLT shear wall tests along with the discussion of the parameters that influence their behavior are discussed. Shake table tests are performed on a two-story platform type construction in three different configurations and a discussion on system response and failure mechanism is presented.
- Chapter 5: Development of the design methodology is presented in this chapter. A discussion on connector design, CLT wall design methodology, and system level requirements are presented.

- Chapter 6: Design requirements for CLT shear wall system based on Chapter 4 and Chapter 5 are presented.
- Chapter 7: CLT archetypes developed in Chapter 3 are evaluated based on the FEMA P695 methodology. Static pushover and dynamic analyses are performed and results presented.
- Chapter 8: Collapse criteria are defined based on which collapse fragilities are developed and archetypes then evaluated. Finally, seismic performance factors for CLT special shear walls are proposed.
- Chapter 9: Summary of the study along with the conclusions and recommendations are presented. Future outlook for the CLT is also briefly discussed.

CHAPTER 2: REVIEW OF CLT AS A LATERAL FORCE RESISTING SYSTEM

With the increasing interest in CLT, many studies that have focused on various aspects of the lateral force resisting system that range 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 investigate seismic behaviour of CLT with the eventual goal of obtaining seismic performance factors or codification of some kind.

A comprehensive research program to investigate the behaviour of 2D CLT wall panels was undertaken at the University of Ljubljana and partially supported by KLH Massiveholz GmbH (Dujic et al., 2005; 2006, 2006a; Dujic and Zarnic, 2006). The purpose of that project 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 condition that include Case A where top of the CLT wall was allowed to translate and rotate, Case B where top of the CLT wall was allowed translation with constrained rotation, and Case C where only horizontal translate 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 condition on overall panel behaviour.

Dujic et al. (2006b,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 and was calibrated using the test data. Calibrated models were the 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 between 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 while the stiffness reduced by about 50%. This study further confirmed that due to the high stiffness and load bearing capacity of the X-lam panels, behavior is governed by the connecting elements.

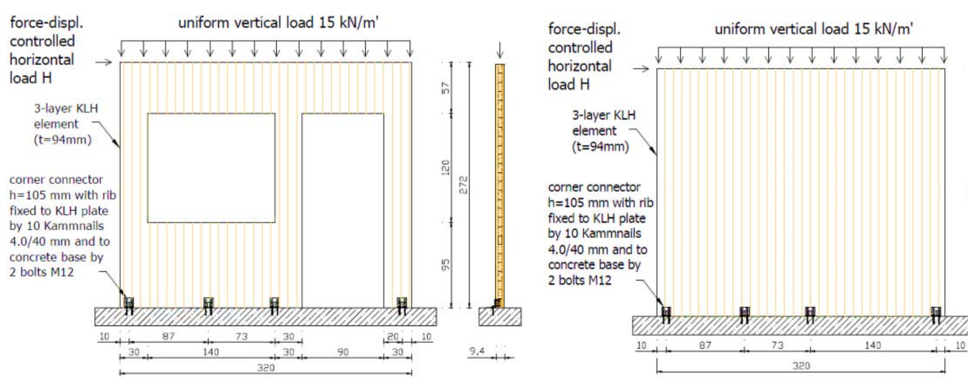
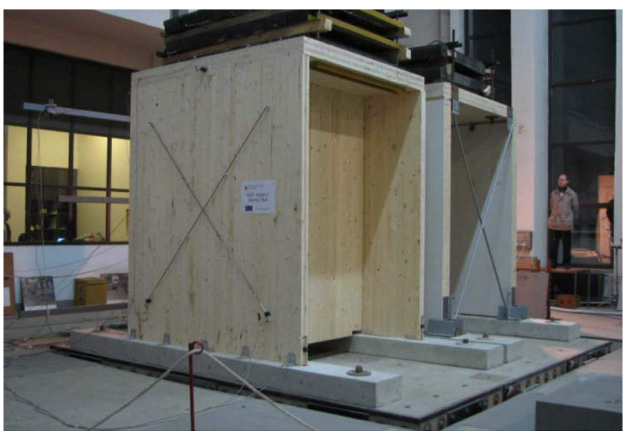


Figure 2.1: CLT wall configuration with and without opening, Dujic et al. (2006b)

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

The fundamental period of vibration was 0.14 sec and 0.28 sec in the longitudinal and lateral directions, respectively. Specimens were subjected to a number of ground motions that including El Centro, Petrovac, Lobe and Friuli and high frequency vibrations (5.0 hz and 7.5 hz). Specimen 2 with the inter-panel 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 inter-panel connector exhibited more ductile behaviour than the specimen with full unit walls. The system demonstrated good correlation with the quasi-static tests.



(a)

(b)

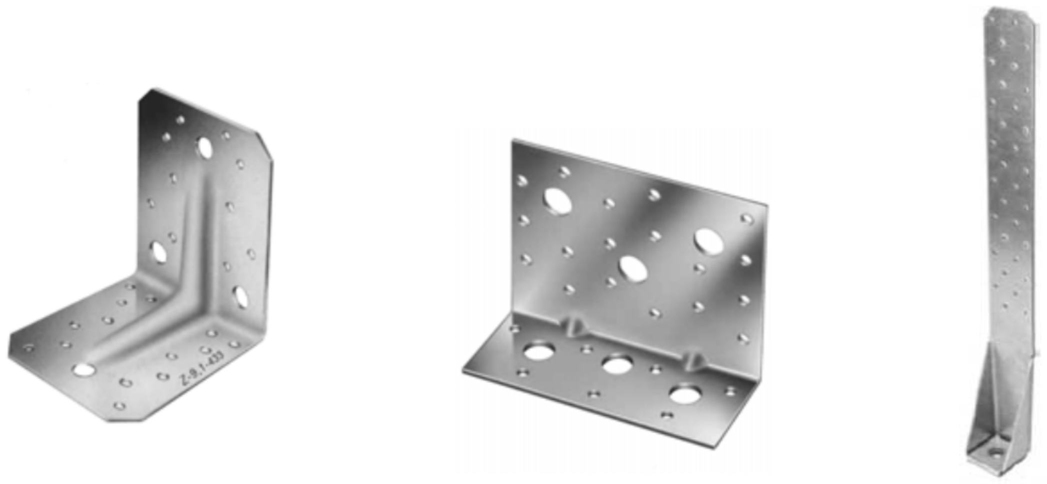
Figure 2.2: (a) Test specimen; (b) Connector for base attachment, Dujic et al. (2006)

The Italian SOFIE project was a multifaceted study whose purpose was 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 in Japan

(NIED), Shizouka University, and the Building Research Institute (BRI) in Japan. The study included tests on various types of connections, quasi-static tests conducted on isolated CLT walls, pseudo-dynamic tests on one story assembly, and full scale shake table tests on a three and a seven-story building (Ceccotti, 2008, Ceccotti et al., 2010).

The results of quasi-static tests and psuedo-dynamic were reported by Lauriola et al. (2006). Quasi-static monotonic and cyclic tests were performed on 9ft 8in. x 9ft x 8in. (2.95m x 2.95m) CLT panels in various configurations under different vertical loadings, with and without openings. All the tests were performed using standard Simpson BMF connector for shear and HTT22 holddowns for uplift except in a couple of cases where custom-made holddowns were utilized. The connectors are shown in Figure 2.3. Annular ringed shank nails of 0.16 in. x 2.36 in. (4mm x 60mm) 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 behaviour. The system exhibited very stiff yet ductile and energy dissipative behaviour that makes it suitable for high seismic region.

Pseudo dynamic tests were conducted on a one-story box type structures with 7m x 7m in plan and 3m in height, shown in Figure 2.4. The specimens were tested under three different configurations which 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 0.15g and 0.50g peak ground acceleration (PGA). The holddowns 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 the wall behaviour is dictated by the connector for low magnitude shear forces.



(a) (b) (c)

Figure 2.3: (a) Simpson BMF-105 angle bracket (b) Simpson BMF-116 angle bracket, (Popovski et al., 2010)
(c) Simpson HTT-22 holddown (Rinaldin and Fragiacomo, 2016)

Figure 2.4: Specimen for psuedo-dynamic testing, third configuration Lauriola et al. (2006)

Full-scale shake table tests on a three story CLT structure were conducted at the NIED Tsukuba, Japan, shake table facility (Ceccotti et al., 2006; 2006b; Ceccotti, 2008, Ceccotti et al., 2013). The three-story test specimen, shown in Figure 2.5, was 23 ft x 23 ft (7m x 7m) in plan with a height of 32ft 10in. (10m). 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 while configuration C was asymmetric. Simpson Strong Tie HTT22 was used at each wall end and openings to resist the uplift and BMF connectors were distributed along each wall to resist the shear loads. Vertical joints were utilized between the walls to achieve a certain level of ductility in the system.

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

Figure 2.5: 3-Story SOFIE Project building (Ceccotti et al., 2006)

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 et al., 2010; Ceccotti et al., 2013). Results from previous phases of the project were used to optimize joint design for X-lam buildings and to obtain ductile response. The building, shown in Figures 2.6 and 2.7, had a plan of 24ft 7in. x 44ft 4in. (7.5m x 13.5m) and a height of 77ft 1in. (23.5m). It was designed considering a q factor of 3 and an importance factor of 1.5 and in accordance with Euro Code 8. According to the Italian National Annex of Eurocode 8 the structure shall be designed for PGA of 0.35g corresponding to the highest hazard level. However, the structure was designed for a PGA of 0.82g in the long direction and 0.60g in the short direction considering the Kobe JMA earthquake. Connections were designed such that ductility and energy dissipation occured in the holddowns, shear connectors, and the inter-panel joints. The specimen was tested only in one configuration and under three earthquakes that included JMA Kobe, Nocera Umbra and Kashiwazaki R1.

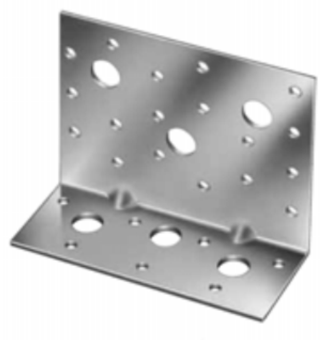
The building had an initial period of 0.43 sec in the short direction (X-direction) and 0.3 sec in the long direction (Y-direction), which is quite short for a 7-story building. A maximum inter-story drift of 2.64 in. (67mm) was observed between second and third floors which was less than 3.15 in. (80 mm) observed in monotonic tests. The structure remained standing throughout all the tests and simple repairs that included tightening loose holddown bolts and replacing connectors were performed between the tests. The system demonstrated self-centering capability and didn't show significant damage up to 0.82g PGA. High acceleration values up to 3.8g 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 et al., 2013).

(a) (b)

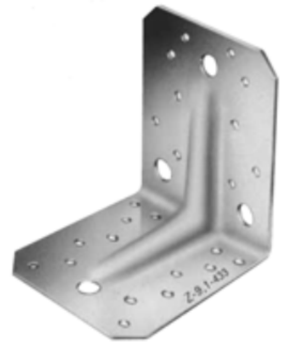
Figure 2.6: 7-Story SOFIE project building; (a) Building Plan (b) Building Elevation; Units given in meters
Ceccotti et al. (2013)

Figure 2.7: 7-story SOFIE structure, Ceccotti et al. (2013)

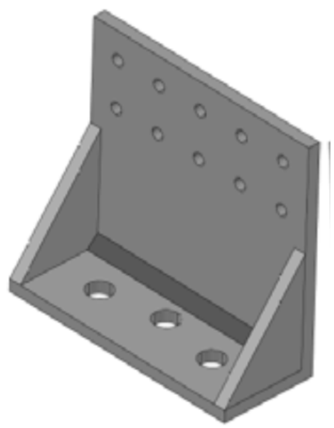
FPInnovations initiated CLT related research in North America through a multi-disciplinary project with the purpose of investigating seismic performance of CLT structures and more specifically to explore seismic modification factors (R_o and R_d factors). Popovski et al. (2010) conducted a total of 32 monotonic and cyclic tests in 12 different configurations that consisted of different aspect ratio panels, openings, walls with inter-panel 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 and are shown in Figure 2.8.



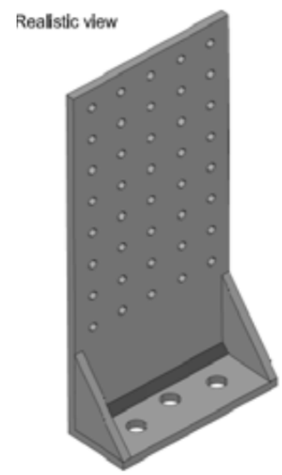
(a)



(b)



(c)



(d)

(e)

Figure 2.8: (a) Simpson BMF-116 angle bracket (b) Simpson BMF-105 angle bracket (c) & (d) Custom made, t=0.25 in. Popovski et al. (2010)
(e) HTT-16 hold-downs* (USP Structural Connectors)

*This Simpson Strong-Tie Currently is currently discontinued

Results of these quasi-static tests verified rigid behaviour of the CLT panels and showed that most of the deformation occurs in the steel brackets and inter-panel connectors. The amount of gravity load had a higher influence on the wall stiffness than its strength. A comparison of the no vertical load case with that of 1.37 kip/ft (20 kN/m) showed a 10% increase in resistance and 28% increase in stiffness in the latter case. This is shown in Figure 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. Holddowns improved seismic performance and step joints (multi-panel configuration) improved ductility.

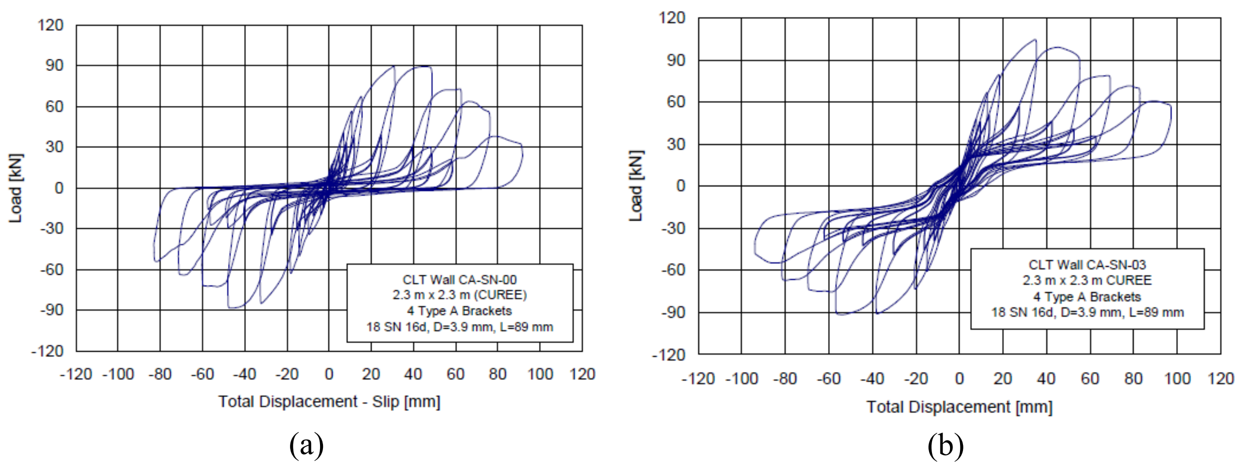


Figure 2.9: (a) No vertical load (b) 1.37 kip/ft (20 kN/m) vertical load, Popovski et al. (2010)

Popovski and Karacabeyli (2012) then used these tests results to perform an AC130 (International Code Council, 2009) seismic equivalency approach in an attempt to quantify seismic performance factors for CLT in the National Building Code of Canada. 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 dame as the R factor in ASCE 7 in the U.S. The results obtained from these quasi-static tests were also used by Pei et al. (2013) to estimate a possible R-factor factor for CLT buildings. This was achieved by investigating CLT wall behaviour using a simplified kinematic model and re-designing the 6-story NEESWood Capstone (van de Lindt et al., 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 systems. However, it should be noted that the study was only performed on a single building, in a specific location, and with limited test data.

To expand upon their initial finding and to better understand CLT system behavior under lateral loads, Popovski and Gavric (2015) performed a number of quasi-static monotonic and cyclic loads on a full-scale two-story structure. The structure dimensions were 19ft 8in. x 15ft 9in. (6.0m x 4.8m) in plan with a total height of 15ft 9in. (4.8m). The structure was constructed in platform type construction and is shown in Figure 2.10. BMF brackets and HTT4 holddowns 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, holddowns and vertical joint 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 each longer and shorter directions of the structure were performed. In order to investigate the effect of additional uplift stiffening and walls perpendicular to the direction of the loading, parameters such as number of holddowns and number of screws in perpendicular wall-to-wall connection were varied, respectively. The CLT structure performed well exhibiting similar failure in both directions. As a result of sliding and rocking of the panels, nail shear failure in the bottom brackets of the 1st story walls were observed and 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 behaviour.

Inter-panel connectors performed as expected and floor diaphragms exhibited rigid behavior even at 3.7in. thickness (94mm). Reducing 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 inter-story drift of 3.2%, mostly sliding deformation, was observed during one of the tests indicating that CLT systems can accommodate larger drift requirements. Test result confirmed that walls perpendicular to the direction of the loading has a significant influence on the behaviour of the building. The authors suggest that while 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.

(a) (b)

Figure 2.10: Two-story CLTstructure (a) E-W direction (b) N-S direction Popovski and Gavric (2015)

Another CLT research project was conducted at the Graz University of Technology, Austria, in collaboration with University of Kassel, Germany (Flatscher et al., 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 215 shear and tension tests were performed on typical connectors used in CLT structures. These tests included angle brackets, holddowns, 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 influence behaviour of the connector.

For the wall tests a total of 17 wall tests were performed in 5 different configurations that included inter-panel connectors and openings. The damping ratio of the third cycle was determined to be 15%-20% which was similar to the values calculated by Lauriola et al. (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 due to shear and bending deformation confirming rigid behaviour of the CLT panel. However, unlike most of the previously discussed studies, test results showed that inter-panel connector 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 et al. (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 due to the gravity load increase in stiffness was more noticeable than increase in strength.

Tsuchimoto et al. (2014) conducted a number static and dynamic tests on the 3-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 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 connection and inter-panel connector. The tests were conducted on walls with and without openings and 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 inter-panel joint exhibited high capacity while failure of inter-panel joint showed higher ductility.

Figure 2.11: 3-Story CLT structure made of Japanese cedar CLT, Tsuchimoto et al. (2014)

In the U.S. a preliminary study on the applicability of post tensioned CLT rocking walls for tall wood building design was completed by Pei et al. (2015). Experimental data is presented in Ganey et al. (2017) which was subsequently used by Akbas et al. (2017) to develop and calibrate numerical models. Six tests were conducted on 1.22m x 4.88m rocking walls and the design parameters of interest were namely, 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 utilization 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 holddowns were used for overturning moment. Increase in applied gravity load resulted in increase in both stiffness and strength, albeit to a different extent and boundary condition was observed to be an influencing parameter. These along with the other findings were considered in conceiving the materials covered in Chapter 4 and 5 that discuss testing and development of the design methodology, respectively.

CHAPTER 3: INDEX BUILDINGS AND ARCHETYPES

Nine main building configurations (i.e. index building models) that consist of single-family dwellings, multi-family dwellings, and commercial (including mixed-use) mid-rise 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 is to verify performance of a class of building configurations but not buildings that can be considered as special cases potentially having unique and irregular configurations which can 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 reduces 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.

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 - 10ft story height
Interior and exterior non-structural wall finishes	- Not considered
Seismic Design Category	- D_{max} and C_{max}/D_{min}
Gravity Load	- ASCE 7 Table C3-1

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

The aspect ratio specified in the table refers to the aspect ratio of individual CLT panels. Longer walls are comprised of multiple high aspect ratio CLT panels connected through inter-panel 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 intensities and building height that have significant influence on the system performance are also considered.

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

Table 3.2. Mapped Spectral Values; FEMA P695

Seismic Design Category	Maximum Considered Earthquake (MCE)			Design Earthquake (DE)
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-Second 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

The following equations are based on ASCE-7 11.4.3.

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

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

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

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

where:

S_{MS} and S_{M1}= MCE response spectral acceleration for short period and 1-second adjusted for site class effect

S_{DS} and S_{D1}= Design Earthquake spectral acceleration for short period and 1-second, respectively.

Short and long period buildings are categorized based on the fundamental period and transition period equations provided in ASCE-7 (2016). If the calculated fundamental period is smaller than the transition period, T_s, the archetype is categorized as short period and if it lager, it is categorized as long period. The equations are given as follows:

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

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

where:

T= Upper limit on calculated period

T_a= Approximate fundamental period, ASCE-7 12.8.2.1

Cu= Coefficient for upper limit on calculated period, ASCE-7 Table 12.8-1

C_t and x = Approximate period parameters, ASCE-7 Table 12.8.2

Ts= Transition period

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

Figure 3.1: Index Building 1

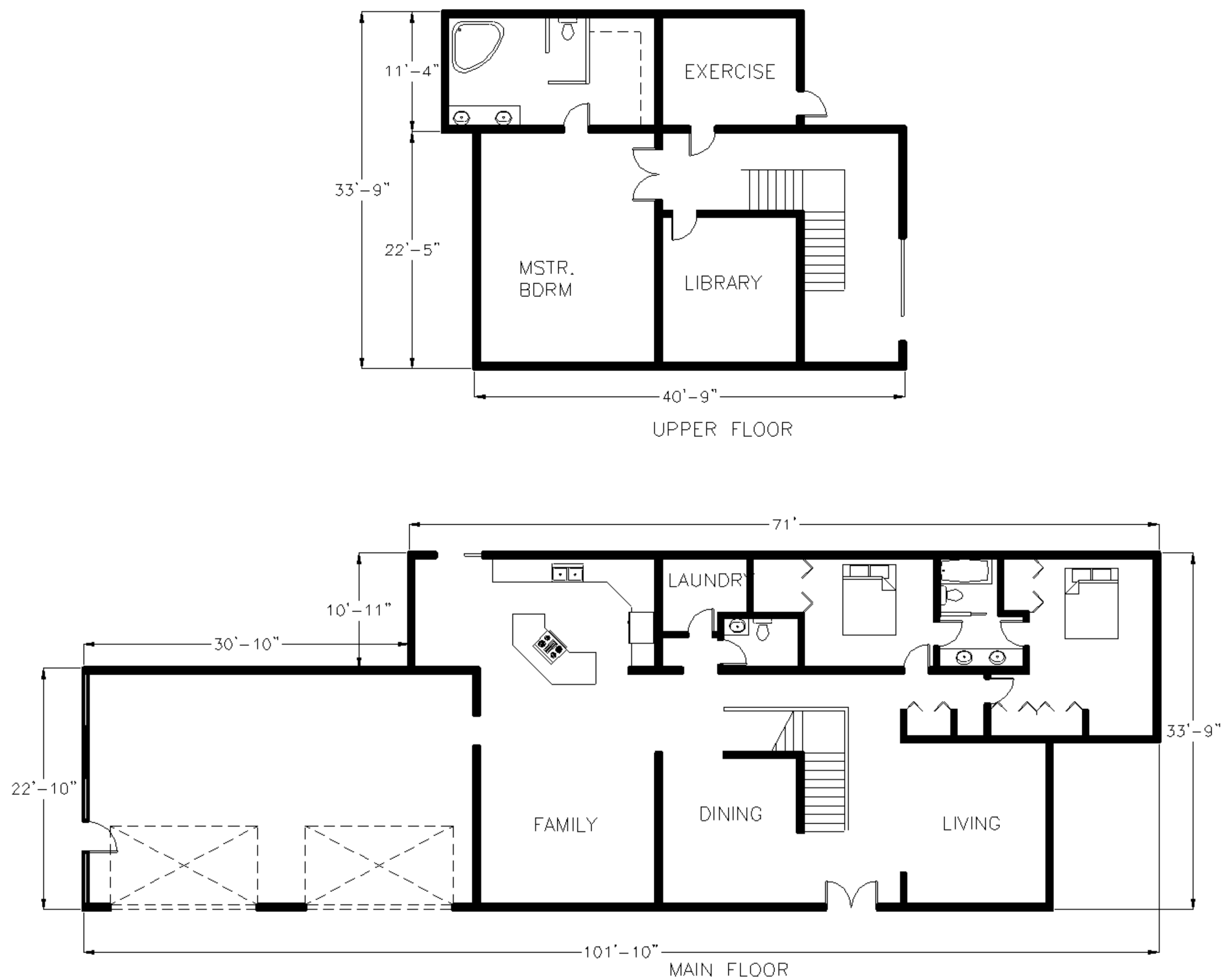


Figure 3.2: Index Building 2

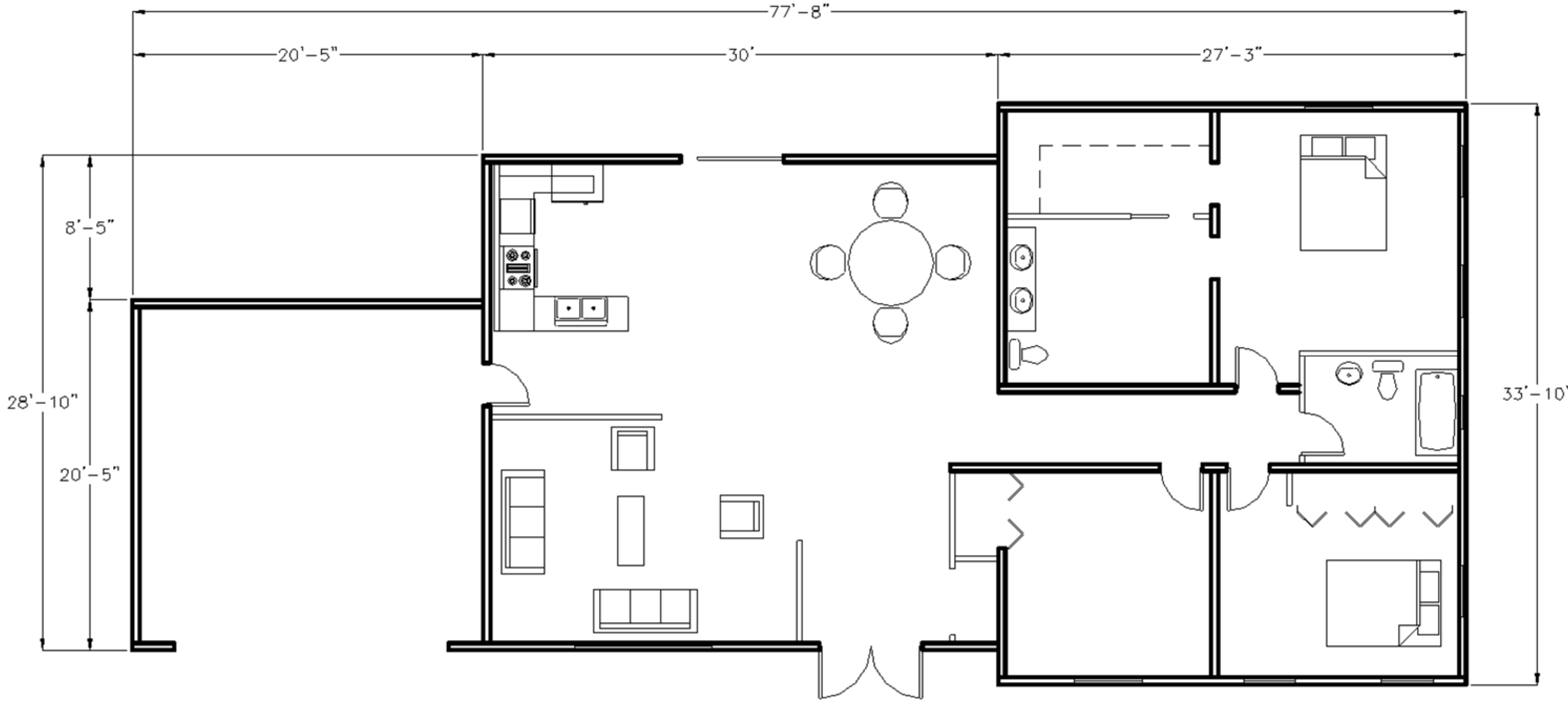
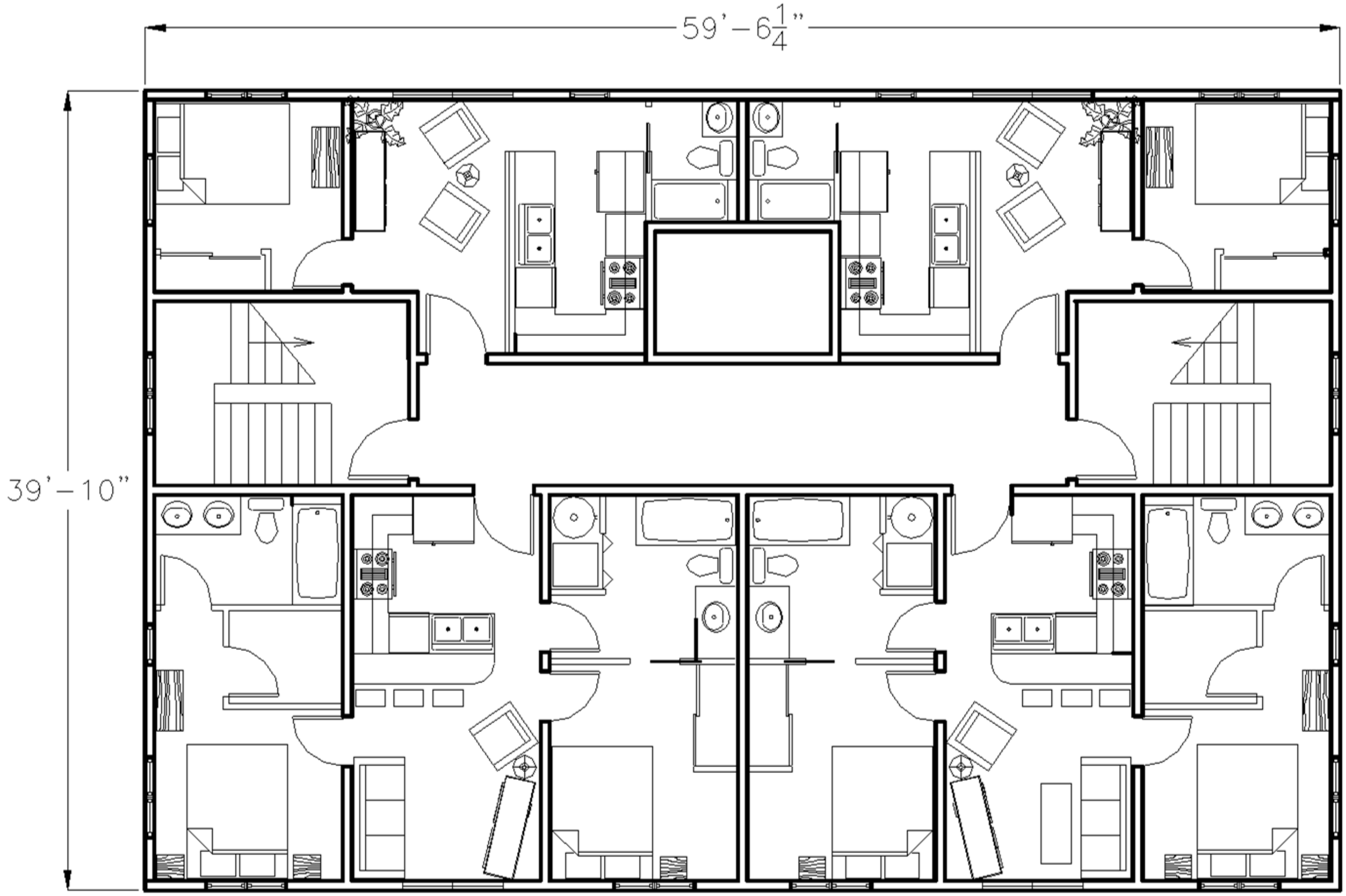


Figure 3.3: Index Building 3



TYP. FLOOR PLAN

Figure 3.4: Index Building 4

Figure 3.5: Index Building 5

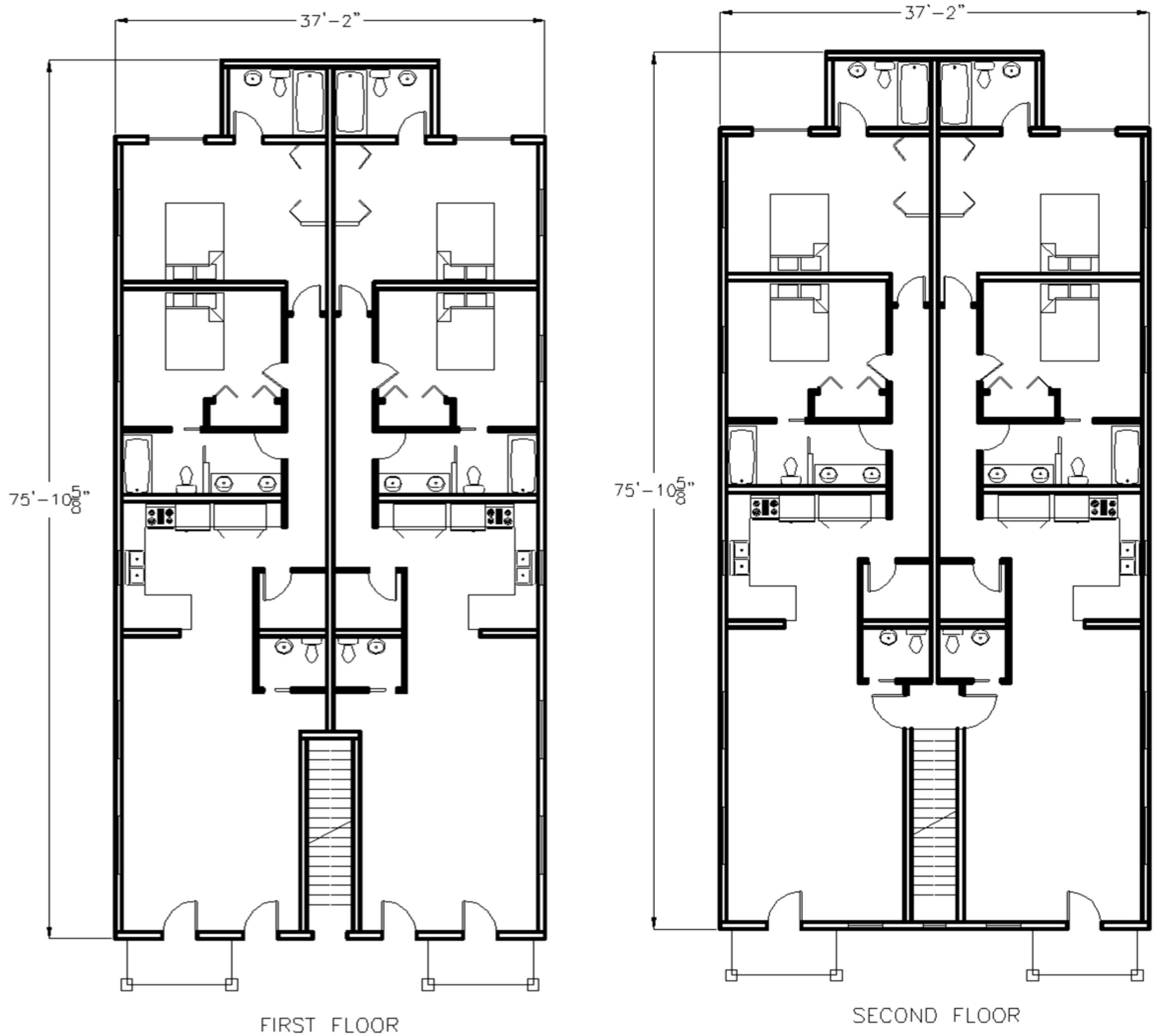


Figure 3.6: Index Building 6

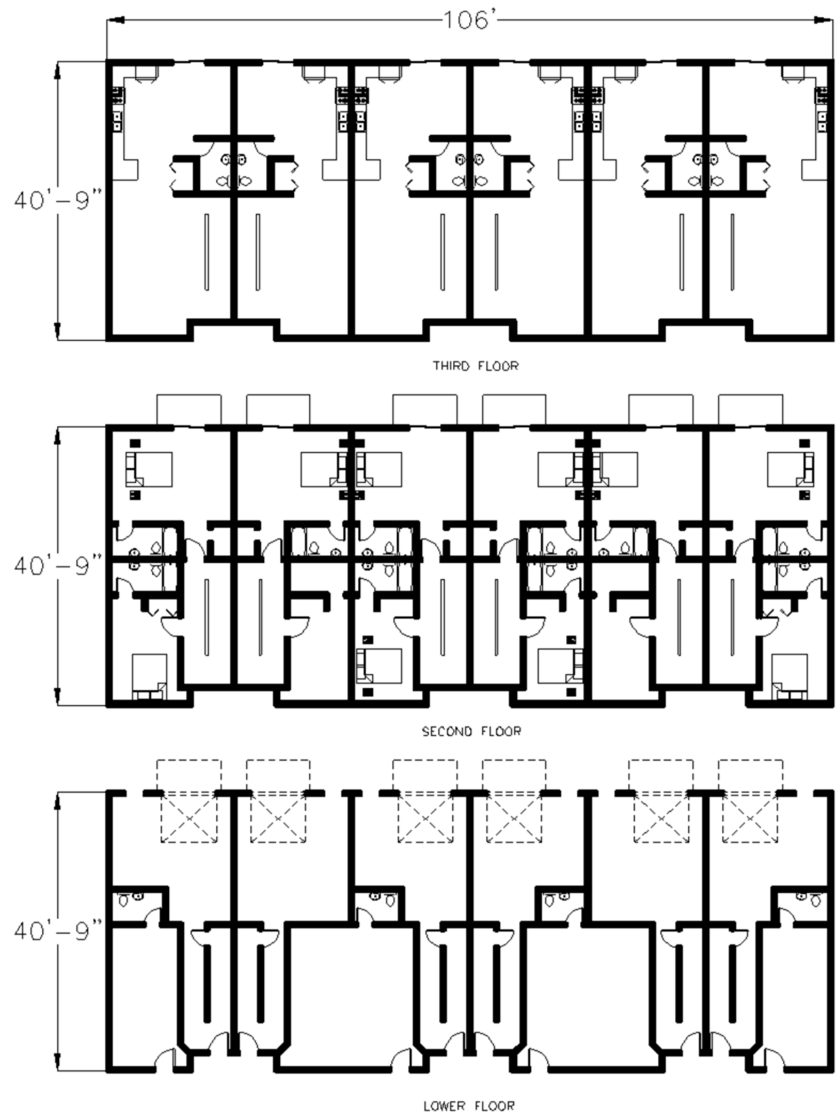


Figure 3.7: Index Building 7

Figure 3.8: Index Building 8

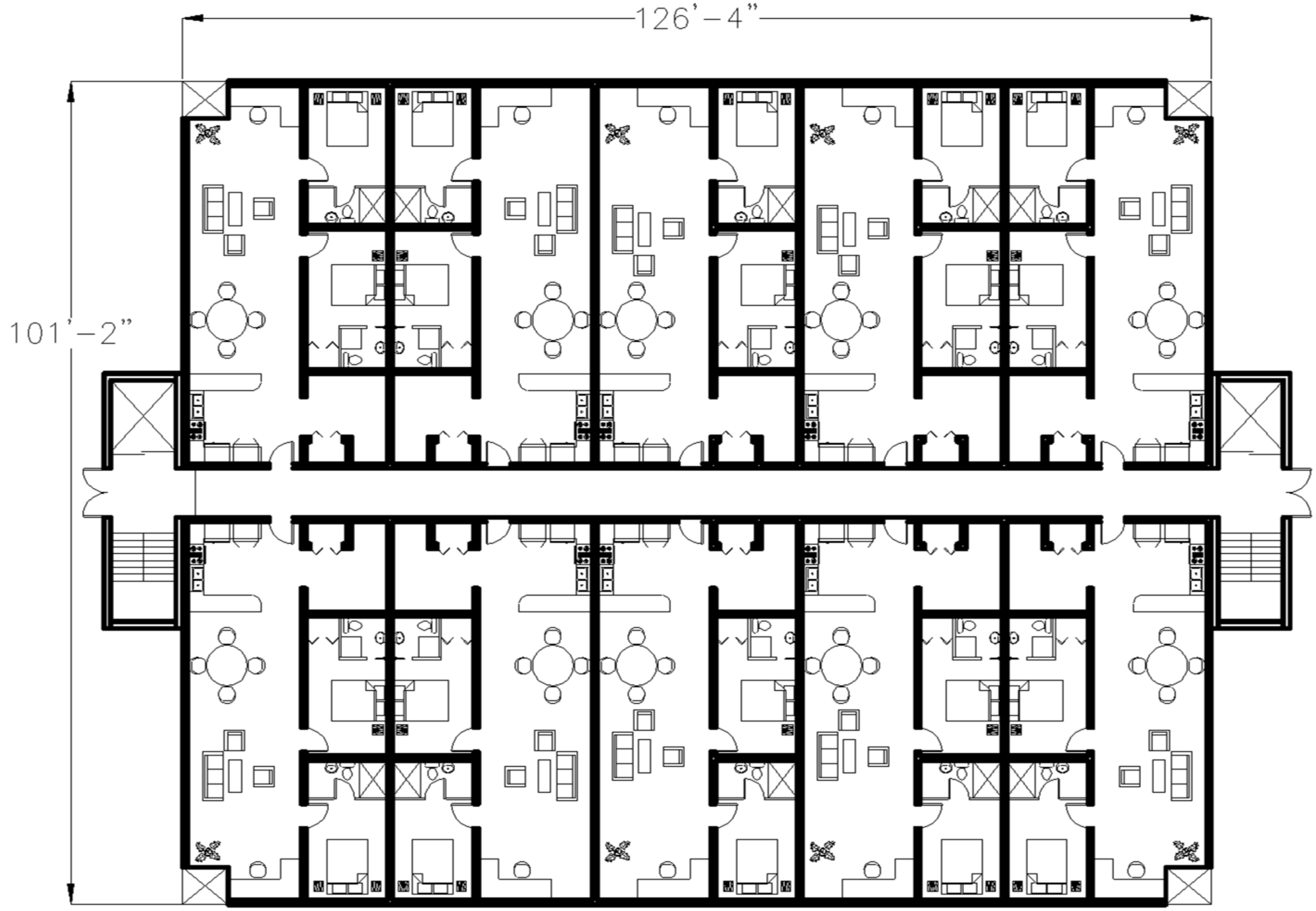
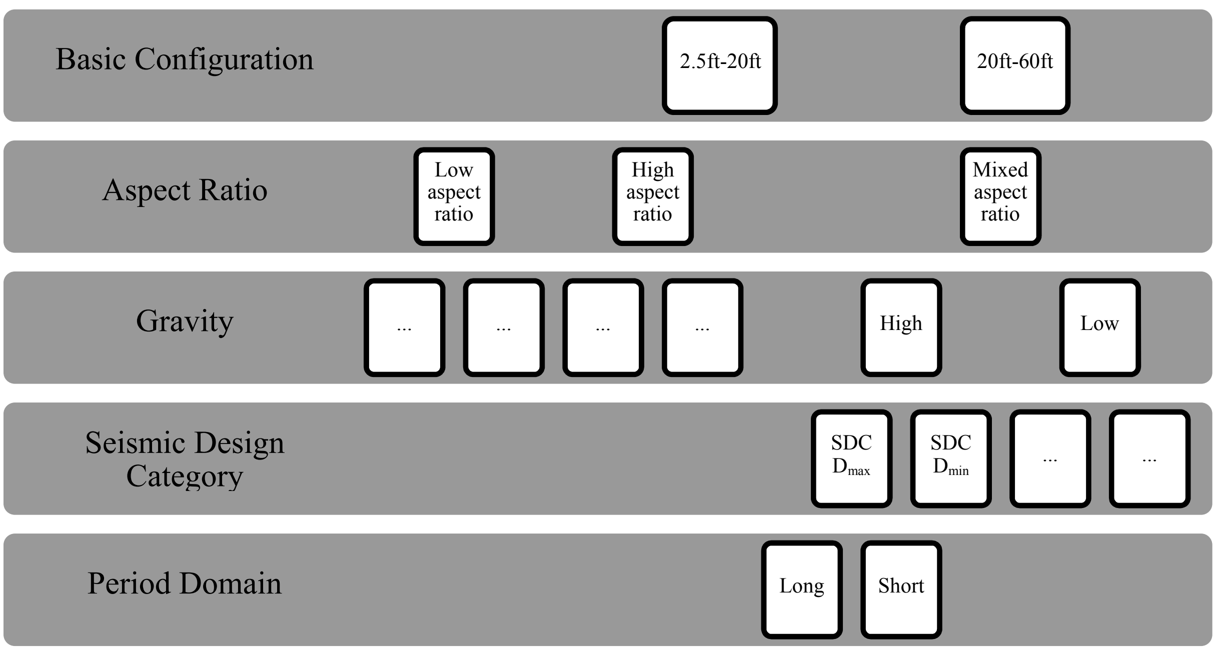


Figure 3.9: Index Building 9

The design space is divided into various performance groups and each one is categorized based on variables such as seismic design category, gravity load, and building height variations. Two basic configurations currently considered for this project are wall lengths (wall lines in a building) of 2.5ft-20ft and 20ft-60ft. Other variables include the aspect ratio of the panels, gravity loads, seismic design category and period domain. Since a number of walls can meet the criteria for a certain performance group, tributary area and the available shear wall length in each case are 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 worse-case scenarios.



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 A. These walls are described as **IndexBuilding_ExtractedWallName_Number of Stories**.

Table 3.3. Extracted Walls

*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_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	1321	x	x
6_1	2	68	47.0	656	x	x
6_2	2	54	54.0	1388	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	1072	-	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	1272	x	x
9_B	6	10	10.0	594	x	x

* Extracted wall description=> **IndexBuilding_ExtractedWallName**
**Available shear wall length=total wall length-openings for doors and windows
*** 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)

Looking at Table 3.3, only four walls shown in Table 3.4 meet the criteria for 2.5ft -20ft wall length and are short period archetypes, their corresponding fundamental period is smaller than T_s. Walls shown in Table 3.5 on the other hand are long period with their corresponding period greater than T_s. Considering the available shear wall length and the tributary area, the highlighted walls in Tables 3.4 and 3.5 are used in the performance groups for the 2.5ft-20ft wall configuration. This is shown in Table 3.6. The performance group matrix in Table 3.6 is populated with the same walls but varying the aspect ratio of the panels, gravity load, and SDC. 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 **R**atio**, HR=H**igh Aspect **R**atio, **MR**= **M**ixed Aspect **R**atio; **LG**=**L**ow **G**ravity, **HG**=**H**igh **G**ravity; **DX**= SDC D_{max}**, DN=** SDC D_{min}**; SP= S**hort **P**eriod**, LP= L**ong **P**eriod

Mixed aspect ratio refers to a configuration where both high and low aspect ratio panels are used in a wall line. For example, 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 multi-plane shear wall. The design methodology presented later specifically states that a multi-panel 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 since wall 9_B cannot be made of mixed aspect ratio configuration (see Appendix B).

Table 3.4. 2.5ft-20ft, short period archetypes

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. 2.5ft-20ft wall, long period archetypes

Extracted Wall	Number of stories	Wall length (ft)	Wall-Opening (ft)	Tributary Area (ft^2)	Low Aspect Ratio	High Aspect Ratio
4_3	4,6,8	19	12.9	392	x	x
4_B	4,6,8	13	13.0	170	x	x
8_3	6,8	14.5	14.5	365	x	x
9_B	6,8	10	10.0	594	x	x

Table 3.6. Performance group matrix for 2.5ft-20ft wall configuration

Group No.		Basic Config.	Grouping Criteria Design Load Level: Gravity	Seis-mic	Period Domain	Archetype description	Archetype No.
PG-1	2.5ft-20ft 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
						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				SDC 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	SDC 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				SDC 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	SDC 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				SDC 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

PG-13		Low	SDC 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			SDC 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	High	SDC 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			SDC 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	SDC 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			SDC 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

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

The highlighted walls in Table 3.8 are 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 be considered more critical in comparison to 4_E; however, in this case the latter is selected in order to avoid having two archetypes (8_B and 8_2) from the same index building in the same performance group.

Since 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), shown in Table 3.9.

Table 3.7. 20ft-60ft, short period archetypes

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	1321	x	x
6_1	2	68	47.0	656	x	x
6_2	2	54	54.0	1388	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	1042	-	x

Table 3.8. 20ft-60ft, long period archetypes

Extracted Wall	Number of stories	Wall length (ft)	Wall-Opening (ft)	Tributary Area (ft^2)	Low Aspect Ratio	High Aspect Ratio
4_1	4,6,8	39.8	25.5	205.2	x	x
4_D	4,6,8	58.6	52.6	781	x	x
4_E	4,6,8	58.6	34.6	600	x	x
8_2	6,8	28.5	23.2	440.5	x	x
8_B	6,8	37.8	16.9	682	x	x
9_1	6,8	42.4	36.4	650.75	x	x
9_3	6,8	46.4	46.4	1272	x	x

Table 3.9. Performance group matrix for 20ft-60ft wall configuration

Group No.		Basic Config.	Gravity	Seismic	Period Domain	Archetype description	Archetype No.
PG-25	20ft-60ft 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
						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				SDC 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	SDC 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				SDC 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	SDC 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				SDC 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

Grouping Criteria: Design Load Level (Gravity, Seismic)

PG-37		Low	SDC 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			SDC 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	High	SDC 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				Long	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			SDC 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	SDC 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			SDC 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

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 reduce the number of archetypes numerically evaluated. For this particular study, the proposed system is intended for seismic design category (SDC) D and the system will be assessed for SDC D_{max} and SDC D_{min}. Once 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 reduce the 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 (2009), preliminary analysis and engineering judgment was used to further reduce the number of archetypes.

Seismic loads are defined in terms of seismic design category (SDC) and occupancy category of the structure. Based on the methodology, structures are considered Occupancy Category I or II receive an importance factor equal to unity. The archetypes are designed for the Design Earthquake (DE) based on the equivalent lateral force (ELF) procedure explained in Sec 12.8 of ASCE Standard 7-16 (2016) and are then evaluated for the Maximum Considered Earthquake (MCE). MCE demand is defined in terms of spectral ordinates presented earlier in the chapter. Preparation of index archetype designs requires selection of initial (trial) values for the response modification factor, overstrength factor, and deflection amplification factor. Trial value for R will be 3 and 4 with C_d=3. In total 9 index buildings and 72 archetypes were considered in this study.

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 are 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
- Assembly and system test data

Material testing was not conducted as part of this study since the data can be obtained from the previous studies. The full test matrix is presented later in this report. CLT material specifications are available from the APA-The Engineered Wood Association in the ANSI/APA PRG 320 (2012) standard that provides information on performance and requirements for Rated Cross Laminated Timber.

Both connector and wall tests were conducted using a test protocol specifically developed for light-frame wood fastener, wall, and other assembly tests developed by Krawinkler et al. (2000). Figure 4.1 presents what has become known as the "CUREE test protocol". The testing protocol is 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 its 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.

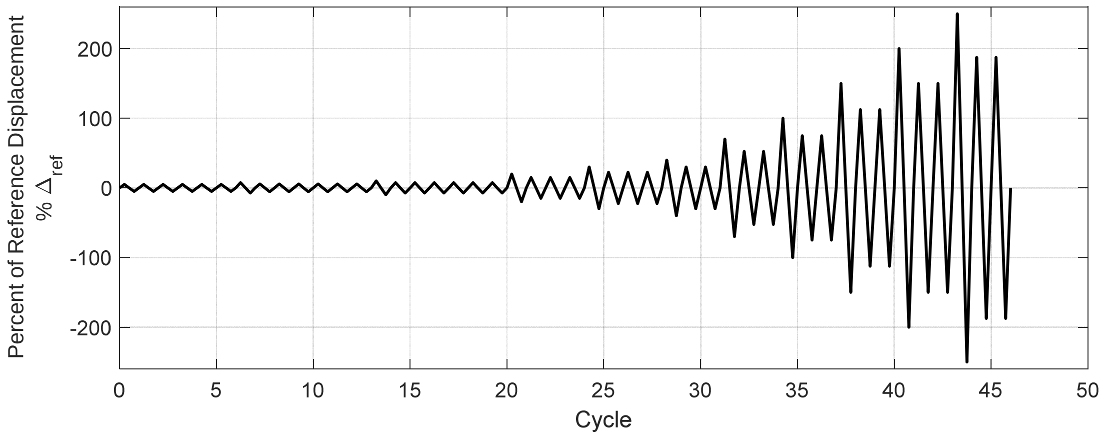


Figure 4.1: CUREE loading protocol (after Krawinkler et al., 2000)

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 inter-panel connectors. Both types of connectors were manufactured from 0.108 in. ASTM A653 (2017) Grade 33 sheet metal (steel) in the structures laboratory at CSU to keep the connector testing as generic as possible. Steel angle brackets and inter-panel connectors are shown in Figures 4.2-4.4. The A3 connector uses eight 16d box nails (3-1/2 in. x 0.135 in.) and B3 connector uses sixteen 16d box nails (3-1/2 in. x 0.135 in.) with bolts prepared from ASTM 1554 Grade A36 steel designed per the National Design Specification (ANSI/AWC, 2015). The metal bracket transfers all the imposed deformation to the nails which are designed to yield under lateral load and eventually pull out of the CLT panel to ensure predictable nonlinear behavior of the fasteners. The inter-panel connector, shown in Figure 4.4, is equivalent to the A3 angle brackets in terms of the number of nails used in the connector. The test configurations for both types of connectors are shown in Figure 4.5-4.7. Steel angle brackets are tested under shear and uplift and inter-panel connectors are tested under shear only. Uplift tests are conducted in a similar manner; however, in this case specimens are 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 PRG320 (2012) are considered in testing. E indicates that parallel layers are E-rated or MSR laminations and V indicates that parallel layers are visually graded laminations. In order to capture statistical variability in the tests, one monotonic and ten cyclic tests are performed for each configuration. The summary of connector tests is provided in Table 4.1.

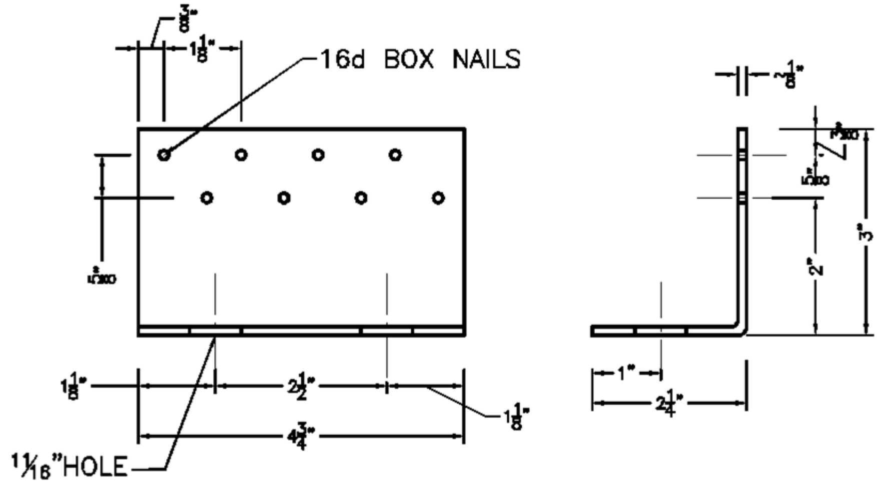


Figure 4.2: A3 type connector

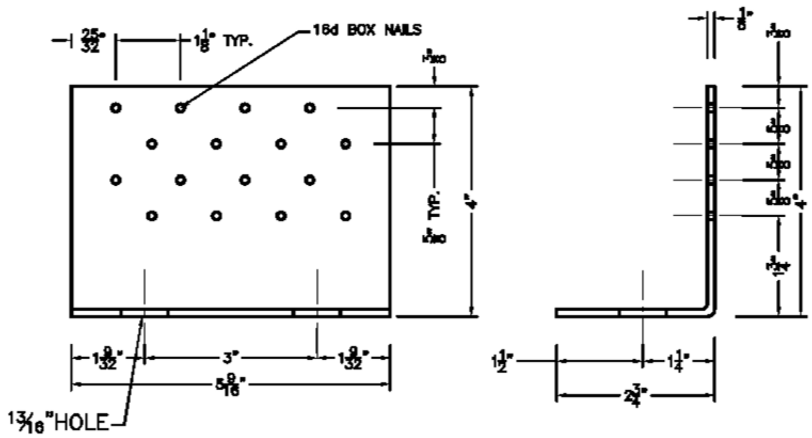


Figure 4.3: B3 type connector

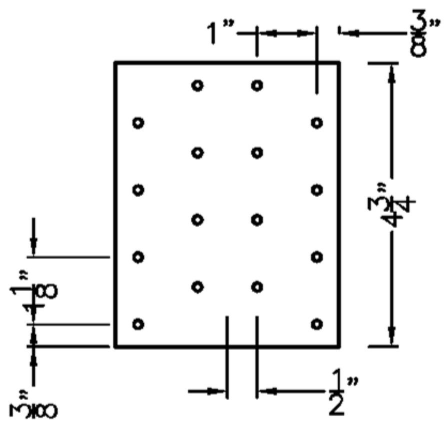


Figure 4.4: Inter-panel connector equivalent to straightened A3 type connector

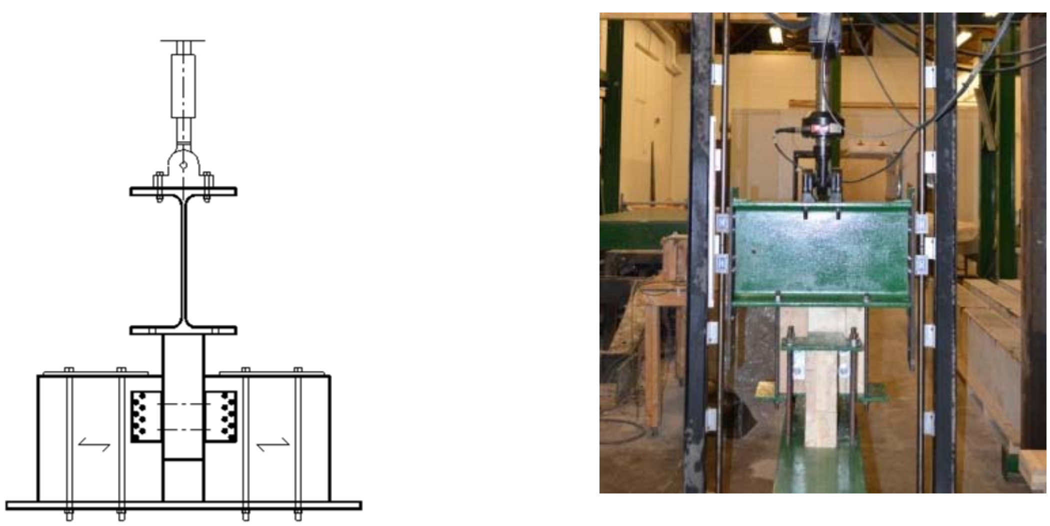


Figure 4.5: Connector shear test

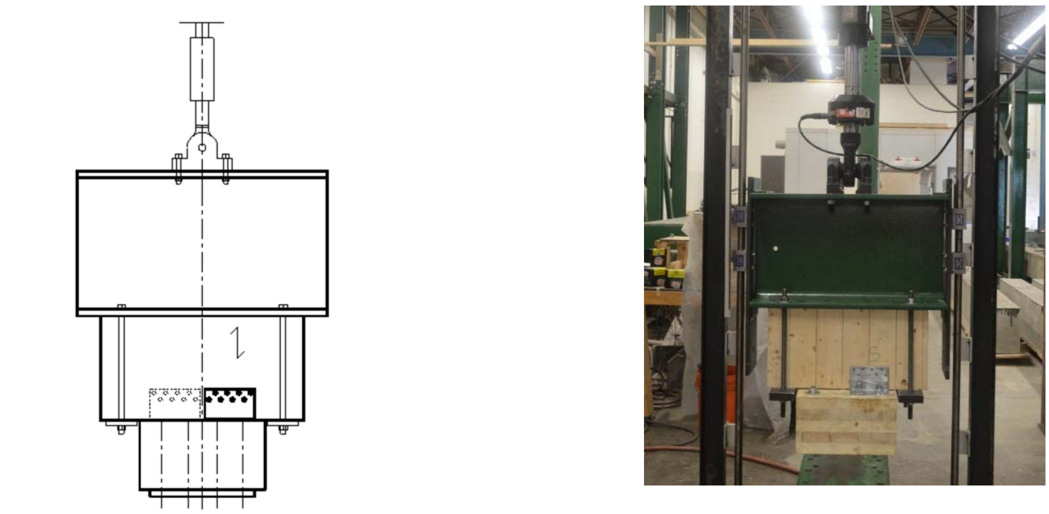


Figure 4.6: Connector uplift tests

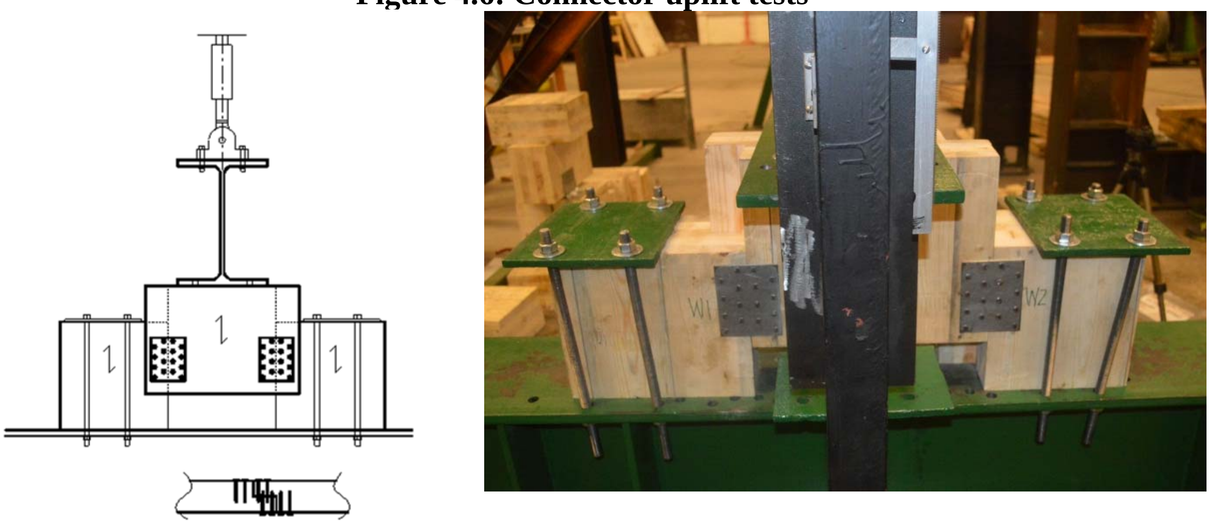


Figure 4.7: Inter-panel connector shear test

4.1.2 Connector testing results

Both connector types, A3 and B3, performed as intended and nail withdrawal was observed, as shown in Figures 4.8-4.11. The testing showed that the nonlinear behavior is primarily that of the fasteners. Monotonic and a sample hysteretic plot for A3 and B3 type connectors are shown in Figures 4.12-4.19 and all the shear and uplift test results are summarized in Table 4.2 and 4.3.

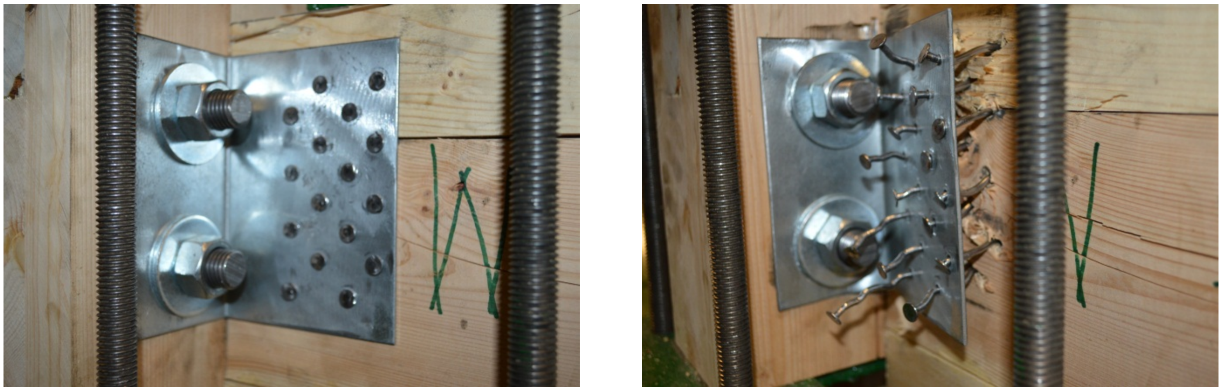
Table 4.1. Connector test matrix

Test Type	Connector type	CLT Grade	Tests
Shear	A3- (8)16d box (3-1/2 in. x 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	
	B3- (16)16d box (3-1/2 in. x 0.135 in.) nails in vertical leg and two 3/4 in. diameter rods (ASTM 1554 Grade A36 steel) in horizontal leg	E1	One monotonic and 10 cyclic
		V2	
Uplift	A3- (8)16d box nails (3-1/2 in. x 0.135 in.) in vertical leg and two 5/8 in. rods (ASTM 1554 Grade A36 steel) in horizontal leg	E1	One monotonic and 10 non-reversed
		V2	
	B3- (16)16d box (3-1/2 in. x 0.135 in.) nails in vertical leg and two 3/4 in. diameter rods (ASTM 1554 Grade A36 steel) in horizontal leg	E1	One monotonic and 10 non-reversed
		V2	
Shear	Inter-panel (8)16d box (3-1/2 in. x 0.135 in.) nails on each side connector center line	E1	One monotonic and 10 cyclic tests
		V2	

(a) Before test

(b) After test

Figure 4.8: A3 type connector before and after shear test



(a) Before test (b) After test

Figure 4.9: B3 type connector before and after shear test

(a) Before test (b) After test

Figure 4.10: A3 type connector before and after uplift test

(a) Before test (b) After test

Figure 4.11: B3 type connector before and after uplift test

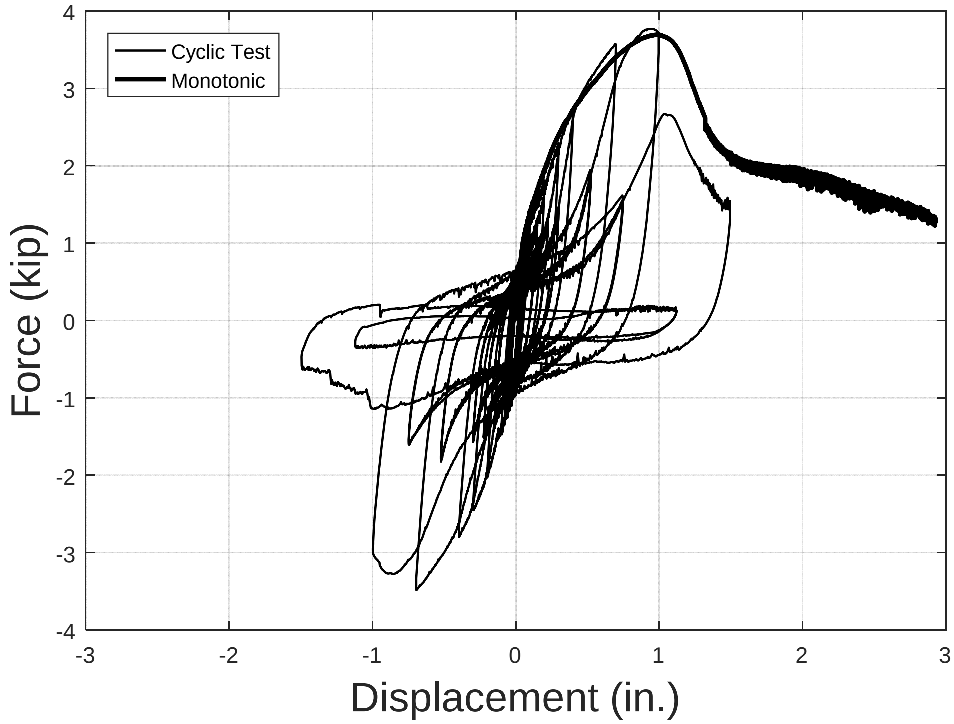


Figure 4.12: A3 connector, Shear Test, E1 grade

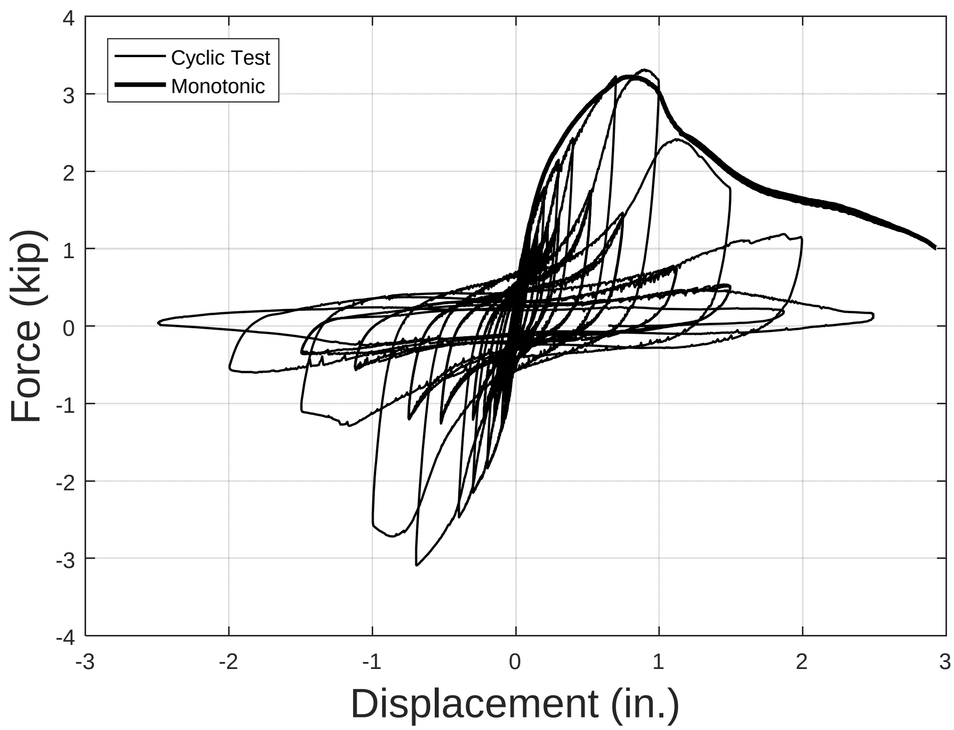


Figure 4.13: A3 connector, Shear Test, V2 grade

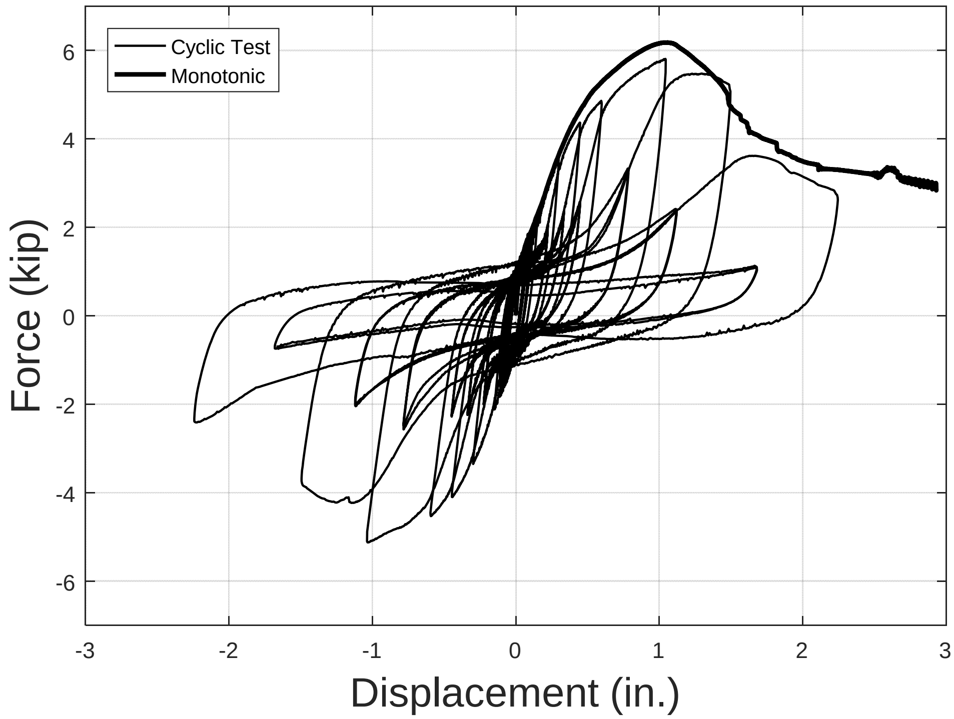


Figure 4.14: B3 connector, Shear Test, E1 grade

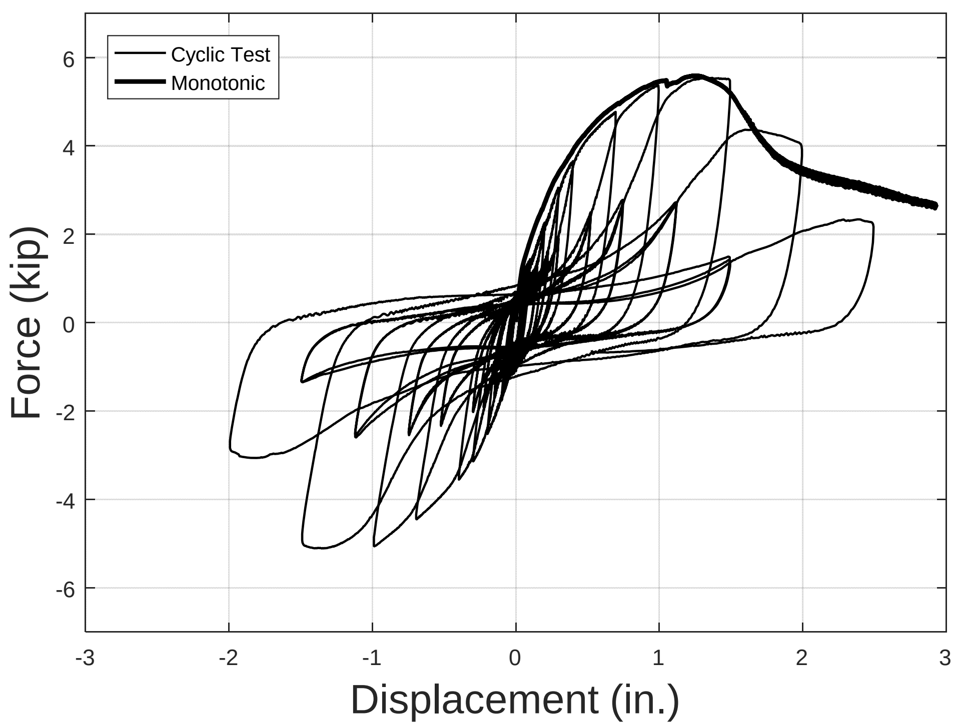


Figure 4.15: B3 connector, Shear Test, V2 grade

Figure 4.16: A3 connector, Uplift Test, E1 grade

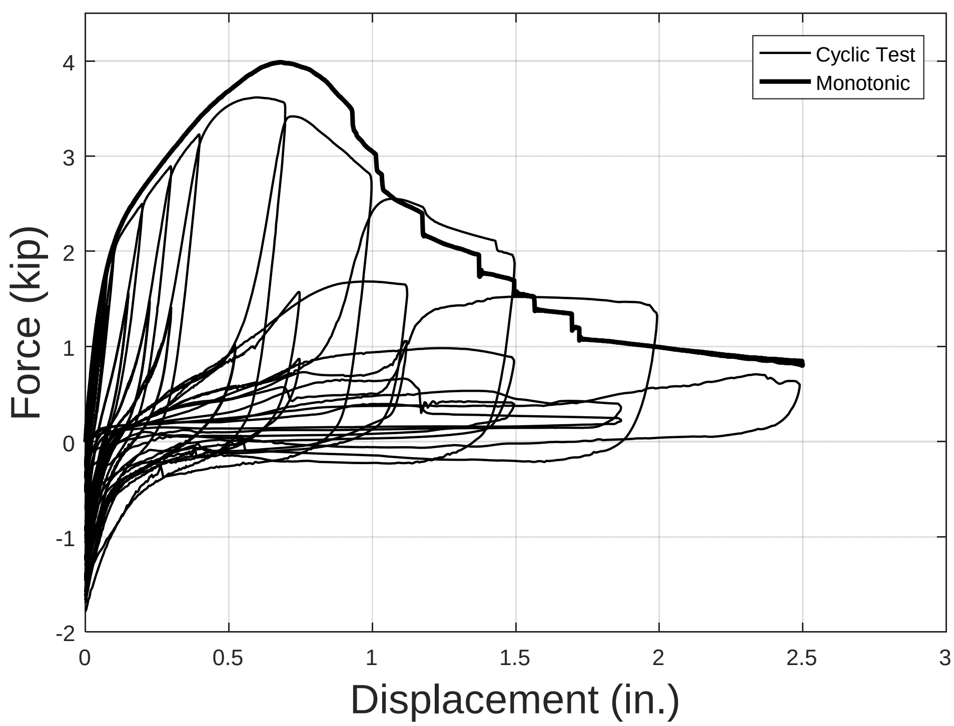


Figure 4.17: A3 connector, Uplift Test, V2 grade

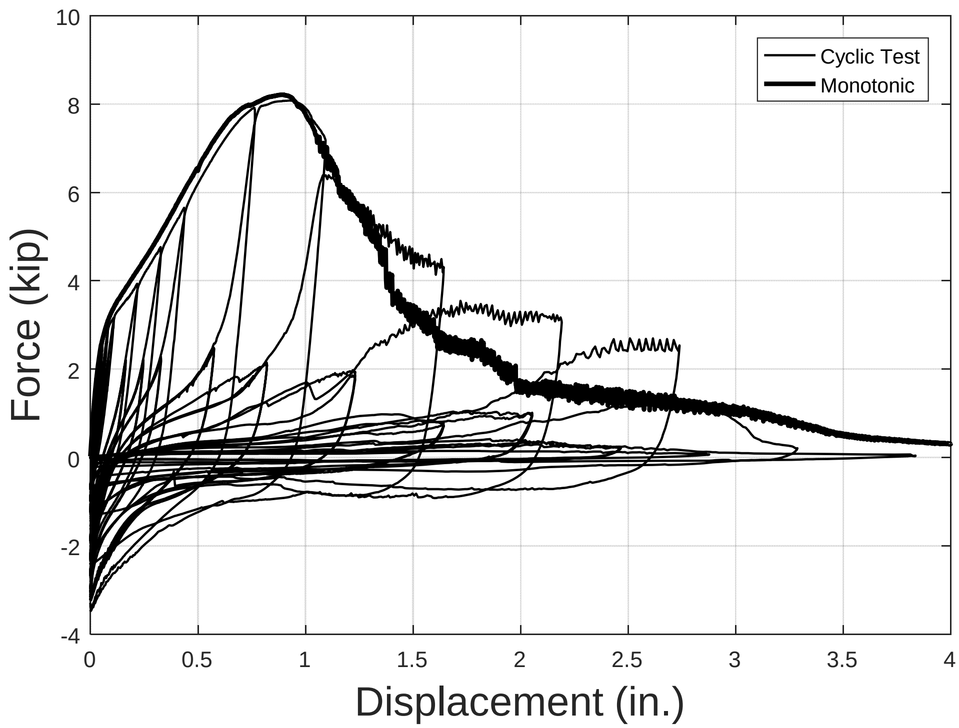


Figure 4.18: B3 connector, Uplift Test, E1 grade

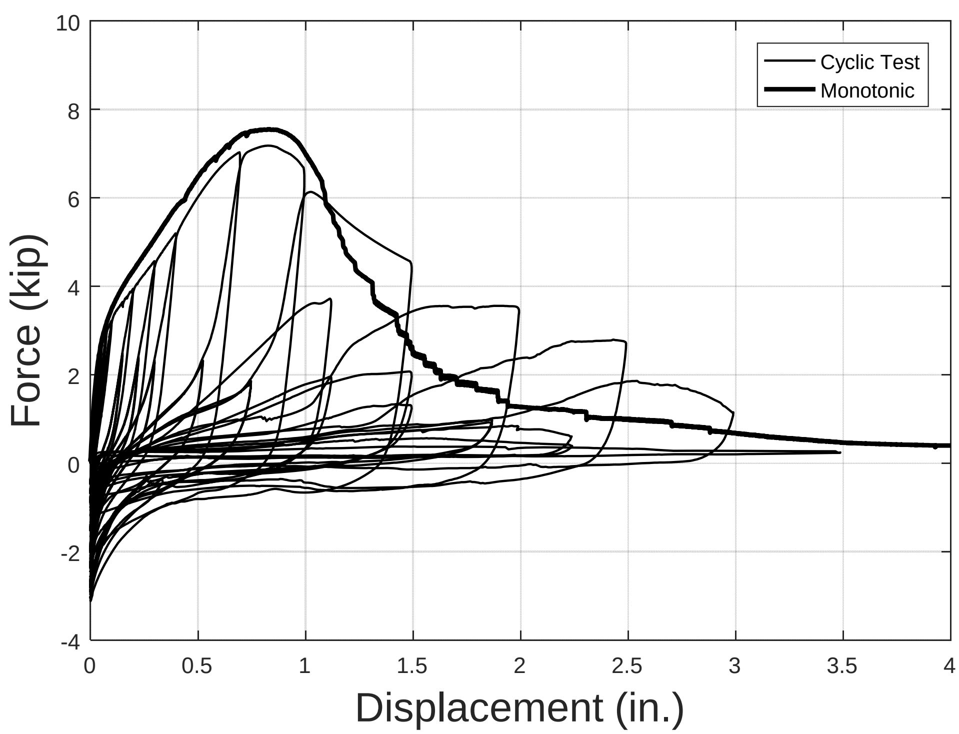


Figure 4.19: B3 connector, Uplift Test, V2 grade

Table 4.2. Connector shear test results

V2 grade

A3 type connector			B3 type connector		
Test #	Max load (kip)	Min load (kip)	Test #	Max load (kip)	Min load (kip)
7	3.31	-3.09	18	5.23	-4.68
8	3.72	-3.06	19	5.92	-5.02
09	3.27	-2.99	20	5.53	-4.95
10	3.34	-3.27	21	5.83	-5.17
11	3.28	-3.01	22	5.63	-4.77
12	3.53	-3.08	23	4.98	-4.53
13	3.40	-3.19	24	5.32	-4.99
14	3.68	-3.30	25	5.29	-4.65
15	3.19	-3.01	26	5.54	-5.11
16	3.35	-2.89	27	5.84	-5.06
Avg.	3.41	-3.09	Avg.	5.51	-4.89
Std. dev.	0.18	0.13	Std. dev.	0.31	0.22
COV	0.053	0.042	COV	0.056	-0.045
95% CI	±0.13	±0.09	95% CI	±0.22	±0.16

E1 grade

A3 type connector			B3 type connector		
Test #	Max load (kip)	Min load (kip)	Test #	Max load (kip)	Min load (kip)
30	3.79	-3.7	40	6.28	-5.28
31	3.84	-3.77	41	6.06	-5.82
32	3.89	-3.9	42	6.28	-5.75
33	3.94	-3.7	43	6.17	-5.77
34	3.69	-3.63	47	6.7	-5.30
35	3.37	-3.33	48	5.92	-5.04
36	4.12	-3.52	49	6.53	-5.60
37	3.76	-3.56	50	6.09	-5.12
38	3.77	-3.48	51	5.81	-5.12
39	4.04	-3.55	52	6.57	-5.88
Avg.	3.82	-3.61	Avg.	6.24	-5.47
Std. dev.	0.21	0.16	Std. dev.	0.29	0.33
COV	0.055	0.044	COV	0.046	-0.060
95% CI	±0.15	±0.12	95% CI	±0.21	±0.23

Table 4.3. Connector uplift test results

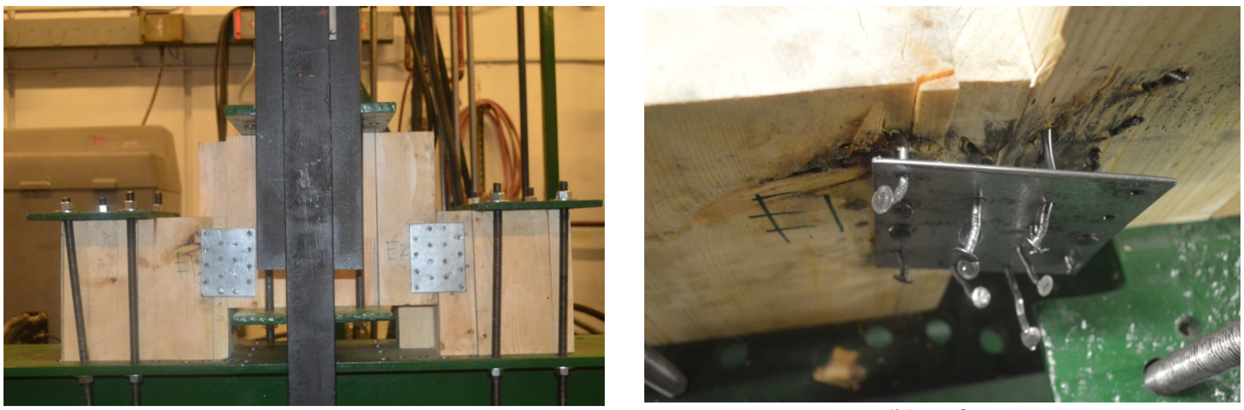
V2 grade

A3 type connector		B3 type connector	
Test #	Max load (kip)	Test #	Max load (kip)
72	3.53	108	7.47
73	3.97	110	8.21
74	4.24	111	7.89
75	3.83	113	7.44
76	3.76	115	7.65
77	3.64	128	7.18
78	3.69	129	7.90
79	4.07	130	7.32
80	3.62	131	7.28
81	3.61	132	7.91
Avg.	3.80	Avg.	7.63
Std. dev.	0.23	Std. dev.	0.34
COV	0.061	COV	0.045
95% CI	±0.17	95% CI	±0.24

E1 grade

A3 type connector		B3 type connector	
Test #	Max load (kip)	Test #	Max load (kip)
61	3.86	118	8.51
62	3.63	119	8.55
63	3.57	120	8.98
64	3.76	121	8.71
65	3.60	122	9.19
66	3.50	123	8.08
67	3.60	124	9.22
68	3.65	125	8.83
133	4.38	126	8.83
134	4.64	127	9.43
Avg.	3.82	Avg.	8.83
Std. dev.	0.38	Std. dev.	0.40
COV	0.099	COV	0.045
95% CI	±0.27	95% CI	±0.28

Similar to the shear tests for A3 and B3 type connectors, testing was done to determine the capacity of inter-panel connector with two different grades of CLT, E1 and V2. A connector test is shown in Figure 4.20 and hysteresis provided in Figure 4.21 and 4.22 for E1 and V2 grades, respectively. A comparison of inter-panel connector and A3 type connector shear tests are provided in Figure 4.23. Summary of the test results are provided in Table 4.4.



(a) Before test (b) After test

Figure 4.20: E1 Connector, Shear test

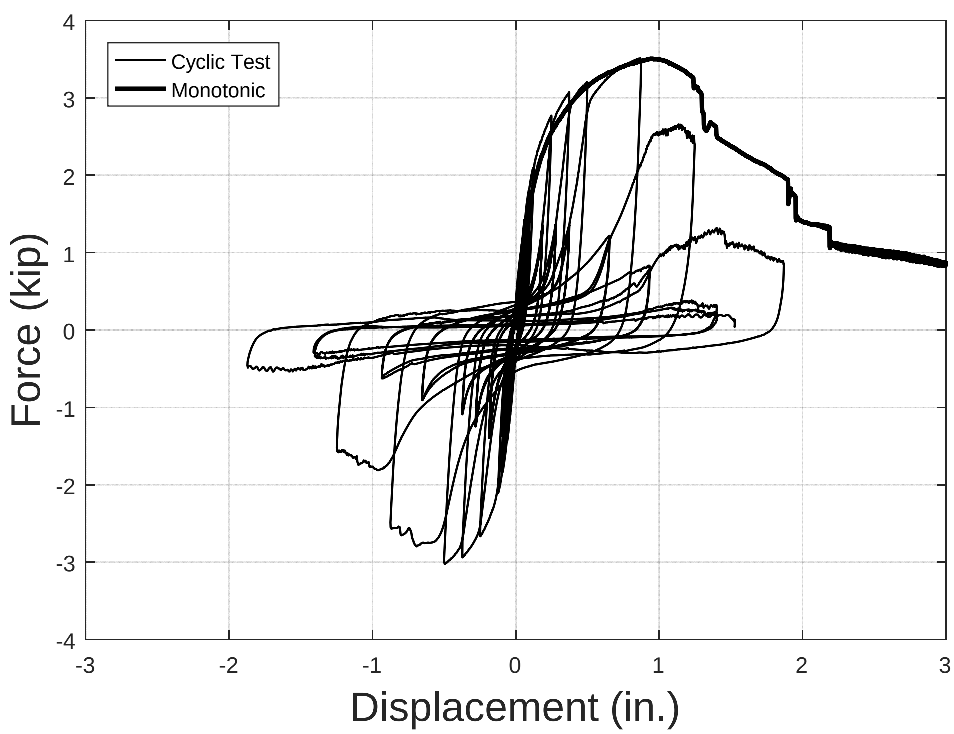


Figure 4.21: E3 connector, Shear Test, E1 grade

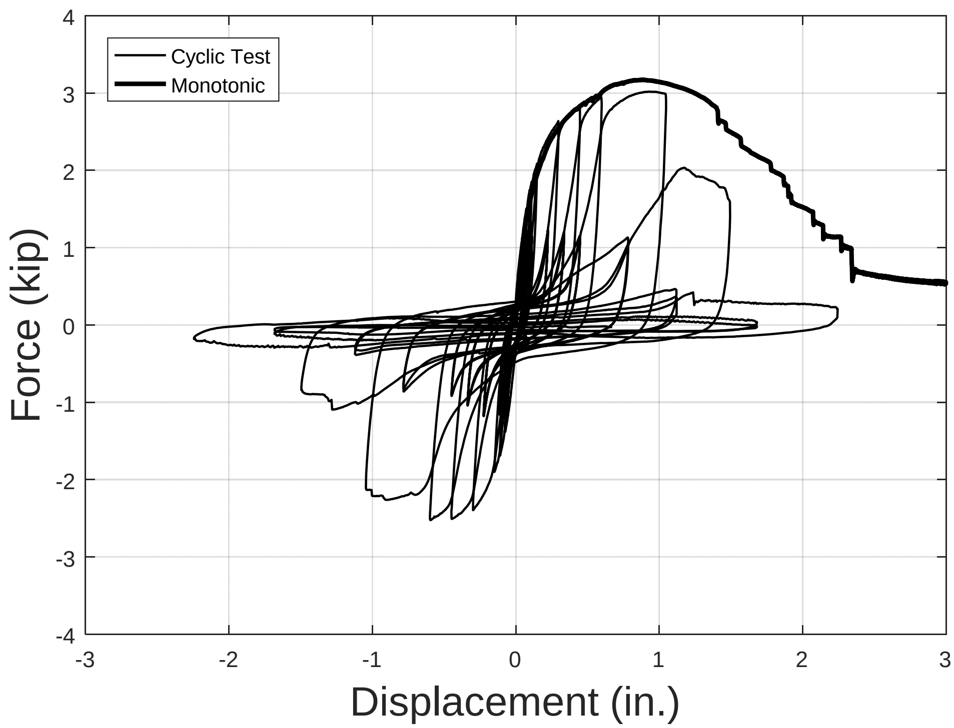


Figure 4.22: E3 connector, Shear Test, V2 grade

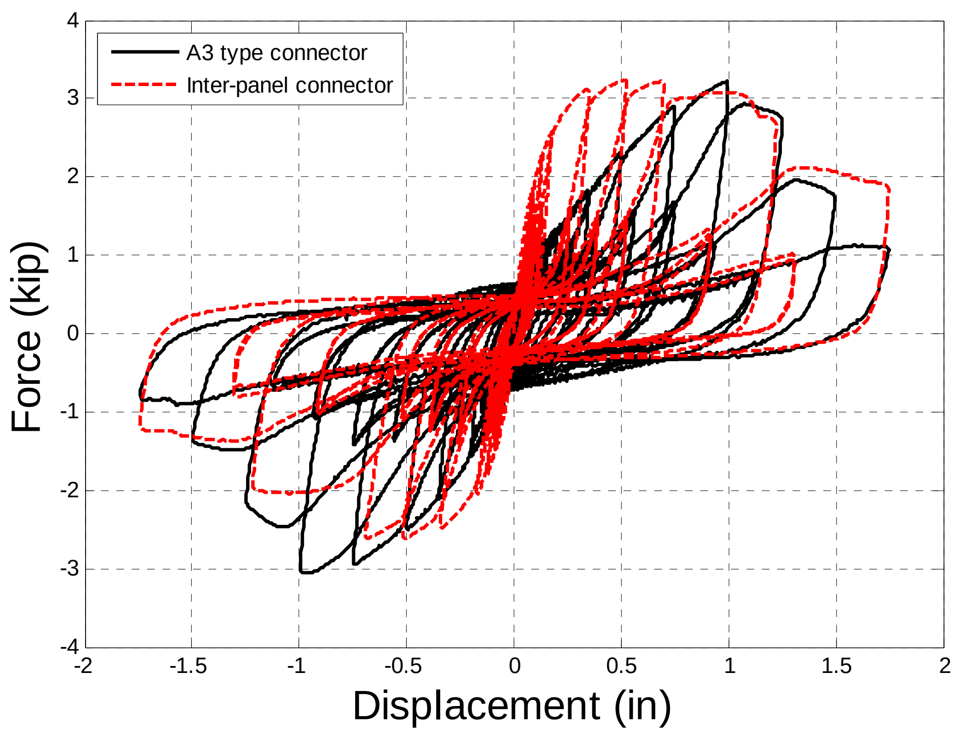


Figure 4.23: Inter-panel connector vs. A3 type connector

Table 4.4. Inter-panel connector shear tests results

E3 type connector

V2 grade			E1 grade		
Test #	Max load (kip)	Min load (kip)	Test #	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

Tests results were analyzed based on the procedure in FEMA P795 (2011) which is similar to ASTM E 2126 (2009) except for the definition of Δ_{yield}. FEMA P795 is the complementary methodology to FEMA P695 and will likely be used for connectors and shear walls to show performance equivalence once the FEMA P695 process for CLT is completed. An example of Test 14 backbone curve is provided in Figure 4.24 and the average parameters for the positive and negative excursions are reported in Tables 4.5-4.9.

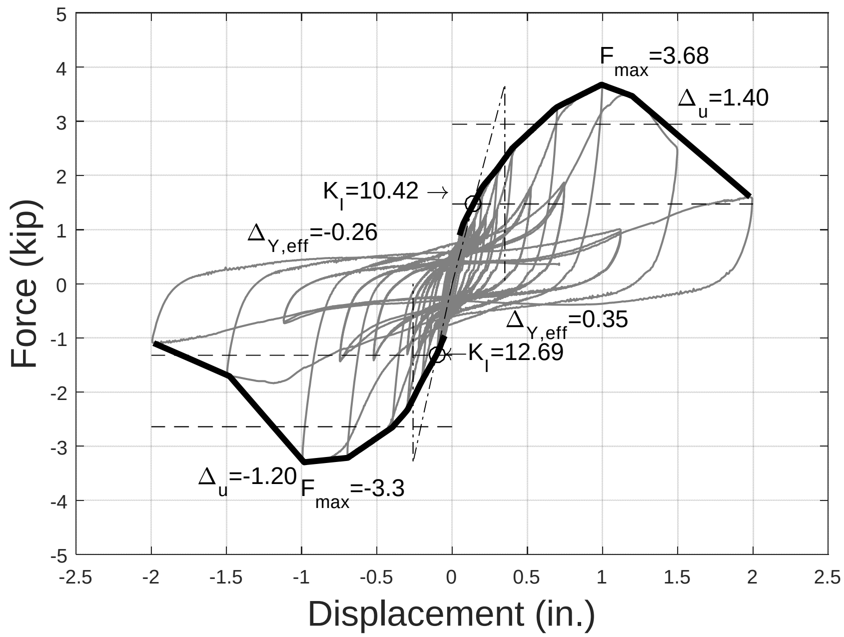


Figure 4.24: Test 14 backbone with parameters calculated based on FEMA P795

Table 4.5. Cyclic envelope parameters for A3 connector shear tests

Test type	Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (0.8 F_{max}) Δ_u(in.)	Effective Ductility Capacity, μ_{eff} (Δ_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
	09	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 4.6. Cyclic envelope parameters for B3 connector shear tests

Test type	Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (0.8 F_{max}) Δ_u(in.)	Effective Ductility Capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Shear Test B3 type connector, V2 grade	18	12.36	0.43	4.95	1.20	1.54	3.58
	19	11.21	0.49	5.46	1.49	1.75	3.57
	20	12.24	0.44	5.24	0.97	1.60	3.68
	21	9.99	0.55	5.50	1.48	2.01	3.65
	22	17.30	0.30	5.20	0.96	1.62	5.38
	23	14.00	0.34	4.74	1.21	1.66	4.88
	24	15.96	0.35	5.15	1.15	1.54	4.45
	25	12.66	0.40	4.96	0.95	1.55	3.92
	26	13.17	0.42	5.31	1.44	1.77	4.27
	27	11.78	0.47	5.45	0.98	1.63	3.49
Avg.		13.07	0.42	5.19	1.18	1.67	4.09
Std. dev.		2.19	0.08	0.25	0.22	0.15	0.64
COV		0.167	0.181	0.048	0.188	0.088	0.157
95% CI		±1.57	±0.05	±0.18	±0.16	±0.10	±0.46
Shear Test B3 type connector, E1 grade	40	11.82	0.50	5.78	0.96	1.39	2.80
	41	14.13	0.44	5.94	0.78	1.41	3.20
	42	16.08	0.38	6.02	0.98	1.49	3.93
	43	15.15	0.42	5.97	0.99	1.55	3.70
	47	14.05	0.44	6.00	0.95	1.53	3.51
	48	14.51	0.38	5.47	1.04	1.44	3.79
	49	14.59	0.42	6.06	1.27	1.90	4.51
	50	16.62	0.34	5.60	1.04	1.31	3.85
	51	14.53	0.38	5.46	1.04	1.45	3.87
	52	13.45	0.48	6.23	1.04	1.52	3.19
Avg.		14.49	0.42	5.85	1.01	1.50	3.64
Std. dev.		1.33	0.05	0.26	0.12	0.16	0.48
COV		0.092	0.119	0.044	0.119	0.107	0.132
95% CI		±0.95	±0.03	±0.19	±0.09	±0.11	±0.34

Table 4.7. Cyclic envelope parameters for A3 connector uplift tests

Test type	Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (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 4.8. Cyclic envelope parameters for B3 connector uplift tests

Test type	Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (0.8 F_{max}) Δ_u(in.)	Effective Ductility Capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Uplift Test B3 type connector, V2 grade	108	53.22	0.14	7.45	0.68	0.92	6.57
	110	49.43	0.16	8.16	0.96	1.15	7.19
	111	40.98	0.19	7.79	0.89	1.11	5.84
	113	52.21	0.14	7.44	0.81	1.02	7.29
	115	42.39	0.18	7.63	0.89	1.27	7.06
	128	41.54	0.17	7.17	0.84	1.13	6.65
	129	38.42	0.20	7.88	0.88	1.23	6.15
	130	58.40	0.13	7.30	0.65	1.12	8.62
	131	42.03	0.17	7.25	0.84	1.16	6.82
	132	53.26	0.15	7.86	0.96	1.40	9.33
Avg.		47.19	0.16	7.59	0.84	1.15	7.15
Std. dev.		6.88	0.02	0.32	0.10	0.13	1.07
COV		0.146	0.125	0.042	0.119	0.113	0.150
95% CI		±4.92	±0.02	±0.23	±0.07	±0.09	±0.77
Uplift Test B3 type connector, E1 grade	118	26.19	0.32	8.51	0.93	1.08	3.38
	119	20.42	0.42	8.53	0.97	1.20	2.86
	120	23.29	0.39	8.97	0.93	1.12	2.87
	121	30.98	0.28	8.68	0.93	1.10	3.93
	122	23.71	0.39	9.19	1.02	1.30	3.33
	123	27.86	0.29	8.08	0.93	1.13	3.90
	124	22.45	0.41	9.21	1.06	1.25	3.05
	125	29.32	0.30	8.80	0.89	1.10	3.67
	126	25.71	0.34	8.81	0.97	1.13	3.32
	127	28.12	0.33	9.42	0.97	1.17	3.55
Avg.		25.80	0.35	8.82	0.96	1.16	3.38
Std. dev.		3.33	0.05	0.40	0.05	0.07	0.38
COV		0.129	0.143	0.045	0.052	0.060	0.112
95% CI		±2.38	±0.04	±0.28	±0.04	±0.05	±0.27

Table 4.9. Cyclic envelope parameters for E3 connector shear tests

Test type	Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (0.8 F_{max}) Δ_u(in.)	Effective Ductility Capacity, μ_{eff} ($\Delta_u/\Delta_{Y,eff}$)
Shear Test E3 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 E3 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

4.2 Isolated wall testing

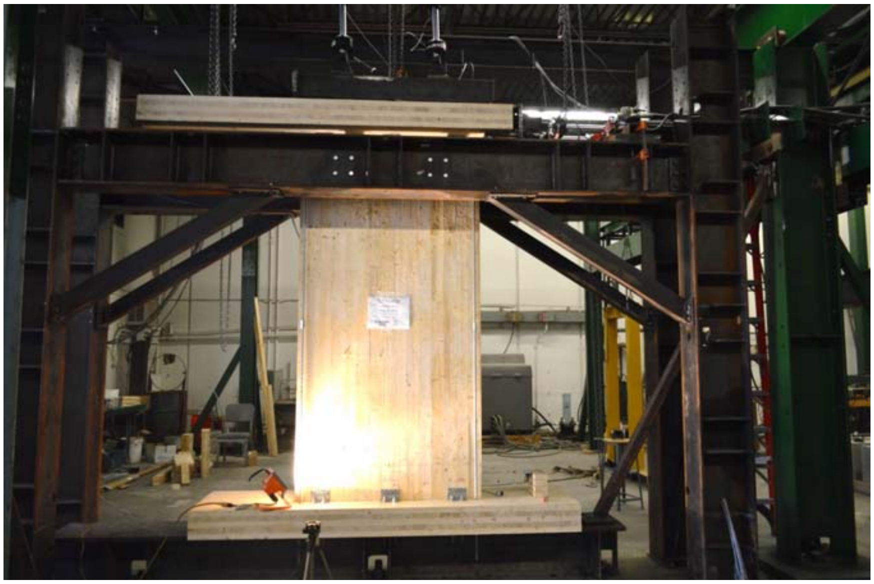
4.2.1 Test setup and loading protocol

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. These parameters are:

- Boundary condition
- Presence of gravity loading
- Connector type
- Connector thickness
- CLT grade
- CLT panel aspect ratio
- Panel thickness
- Presence of inter-panel connector (vertical joint)

The main design assumption for these walls as dictated by the design approach is that all overturning is resisted by overturning anchor (tie rod or holddowns) at wall ends and the shear is resisted by the angle brackets. This assumption was also adopted in the initial stages of the SOFIE project (Ceccotti, 2008) and it aligns well with the already established design method for light-frame wood shear walls. A comparison of different analytical models with experimental data (Gavric et al., 2015) showed this approach to be conservative.

A photo of the general test setup is shown in Figure 4.25. Vertical actuators under load control are used for gravity load and horizontal actuators under displacement control are used for application of the shear load. Since the vertical actuators are in force control, the change in angle will 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 results and the walls restoring force adjusted on the hysteretic plot. The top CLT panel is used as the loader bar to simulate the floor diaphragm and the bottom CLT is used as the base. Lateral braces are provided (not shown in Figure 4.25 for clarity) to prevent out-of-plane movement of the panels. The CUREE loading protocol was used for all the reverse cyclic loading with the reference displacement, Δ_{ref}, of 1 in. and is shown in Figure 4.26. Loading is applied at a rate of 0.033 Hz (30 sec cycles). While the cyclic frequency rate used is slower than recommended in ASTM E2126 to avoid inertial effects, the loading rate for the loading protocol falls within the recommended values of 0.04-2.5 in./sec. This slower cyclic frequency rate is considered to meet the standard as a slower cyclic frequency rate will also avoid any inertial effects.



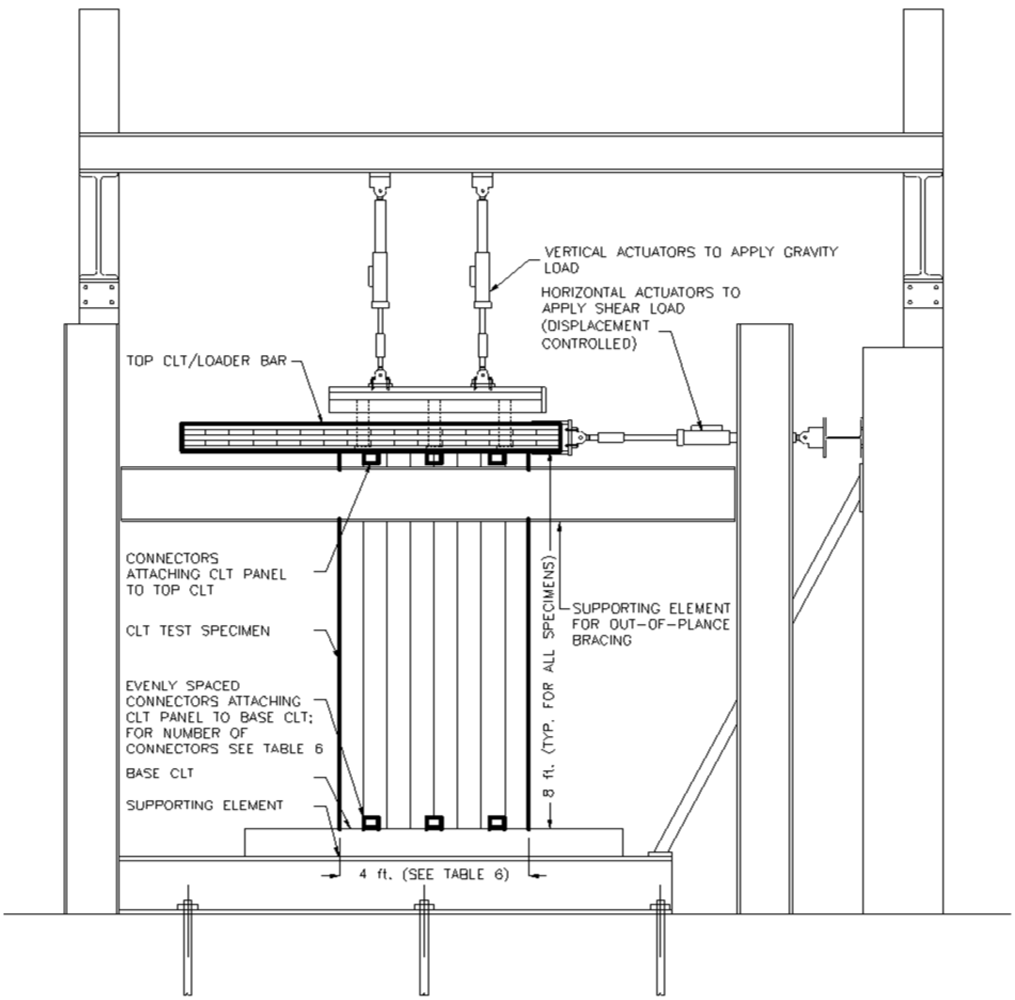


Figure 4.25: Isolated wall test setup

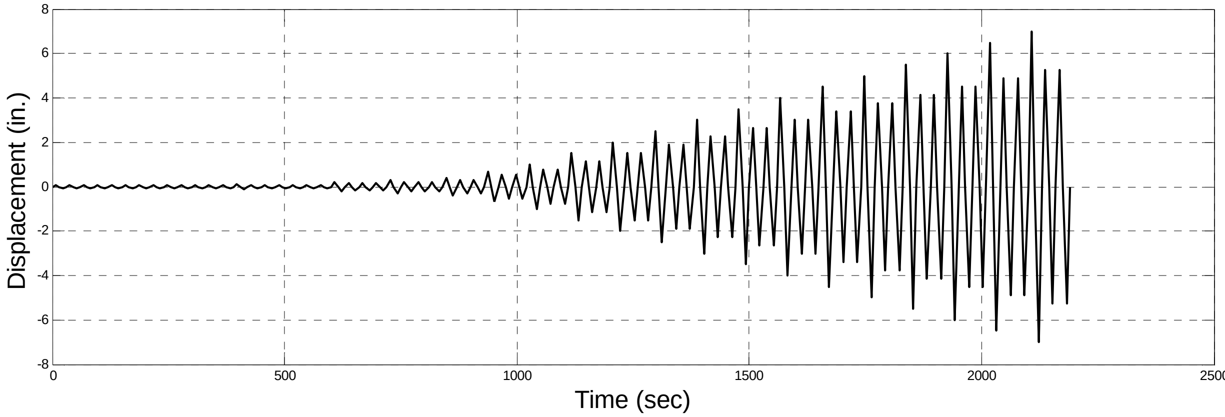


Figure 4.26: Isolated wall test loading protocol

Table 4.10 provides information CLT wall tests and includes CLT grade, geometry, vertical load, and the applied boundary condition*.

Table 4.10. CLT wall matrix

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)	Holddown rod Dia. (in.)
01*,**,***	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
02*,**,***	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
03	V2	8	4	5	6.65	3	A3	0.68	(2) 5/8
04	V2	8	4	5	6.65	3	A3	1.28	(2) 5/8
05	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
06*	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
09	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****	V2	8	8	3	3.9	4	A3	-	(1) 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
27*****	E1	8	4	5	6.89	3	B3(3/16 in.)	-	(2) 3/4
28	E1	8	4	5	6.89	2	B3 (10 gauge)	-	(2) 5/8

*Test 01,02, and 06 were performed with the imposed boundary condition. The imposed boundary condition is explained in detail in Section 4.2.3.3
** Test 01 and 02 were performed during the exploratory phase of the A type connector thickness

*** The top and bottom CLT panels matched the CLT wall panel grade in all testing, except for Test 01 and 02 where the wall panels were of V2 grade while the top and bottom CLT panels were of E1 grade. Top and bottom CLT panels of E1 and V2 grades were 6.89 in. and 6.65 in. in thickness, respectively.
**** Test 22 was (8ft x 8ft) 1:1 aspect ratio wall that is not covered by the design methodology
***** Test 27 and 28 were performed during exploratory phase of B type connector thickness

4.2.2 Test configurations and results

Summarized peak load data are provided in Table 4.11. CLT wall tests results were also analyzed based on the procedure in FEMA P795 (2011) which is similar to ASTM E 2126 (2009) except for the definition of Δ_{yield}. The average parameters for the positive and negative excursions are reported in Table 4.12.

Table 4.11. CLT shear wall testing summarized load data result

Test #	Length (ft)	No. connectors	Connector type	Fmax (kip)	
				+ve	-ve
01	4	3	A1	12.29	10.15
02	4	3	A1	11.89	10.73
03	4	3	A3	14.79	14.62
04	4	3	A3	15.70	14.22
05	4	3	A3	17.90	18.15
06	4	3	A3	19.51	18.02
09	4	3	A3	15.11	11.62
10	4	4	A3	15.20	13.12
11	4	2	A3	8.03	8.70
13	4	2	A3	7.60	10.72
14	4	3	A3	19.98	16.52
15	4	2	A3	10.16	11.02
17	4	4	A3	24.50	21.90
18	4	2	A3	6.74	7.30
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)*	4	A3	13.34	13.84
24	4	2	B3	18.58	18.62
25	4	3	B3	27.80	29.24
26	4 (2)**	8	A3	23.72	24.30
27	4	3	B3(3/16 in.)	22.94	23.64
28	4	2	B3 (10 gauge)	19.17	20.17

* Multi-panel configuration, two 2ft panels ** Multi-panel configuration, four 2ft panels

Table 4.12. CLT shear wall testing cyclic envelope parameters

Test #	Initial Stiffness K_I (kip/in.)	Effective Yield $\Delta_{Y,eff}$ (in.)	Ultimate Load F_{max}(kip)	Displacement corresponding to ultimate load Δ_{Fmax}(in.)	Ultimate derformation (0.8 F_{max}) Δ_u(in.)*	Effective Ductility Capacity, μ_{eff} (Δ_u/ $\Delta_{Y,eff}$)
01	7.96	1.46	11.30	2.44	2.50	1.71
02	6.41	1.79	11.42	2.47	2.60	1.46
03	6.70	2.20	14.71	3.72	4.13	1.88
04	6.45	2.32	14.96	3.71	4.03	1.73
05	8.52	2.13	18.04	4.83	6.00	2.82
06	6.58	3.03	19.12	4.21	5.88	1.94
09	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
22	5.48	2.56	13.99	2.98	3.45	1.35
23	5.67	2.51	14.14	4.74	7.00	2.79
24	9.98	1.87	18.60	5.42	6.00	3.21
25	9.77	2.92	28.43	5.22	5.00	1.72
26	16.21	1.48	24.03	4.47	6.50	4.39
27	10.38	2.25	23.24	4.34	4.75	2.12
28	6.32	3.11	19.55	6.08	6.75	2.17

*In the cases where no descending branch was observed, Δ_u was taken as the maximum deformation executed in the test

Table 4.13 presents general information and explains the purpose of the attached instrumentation. The placement of these instrumentations for a single panel and multi-panel configuration are shown in Figures 4.27 and 4.28.

Table 4.13. Descriptions of instrumentations

Instrument No.*	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 closes 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 closes 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		Inter-panel slip	Displacement (in.)
SP10		Inter-panel slip	Displacement (in.)
SP11	Diagonal along the panel	Panel deformation, added after Test 07	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

*SP= String Potentiometer, LC= Load Cell, SG= Strain Gauge

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

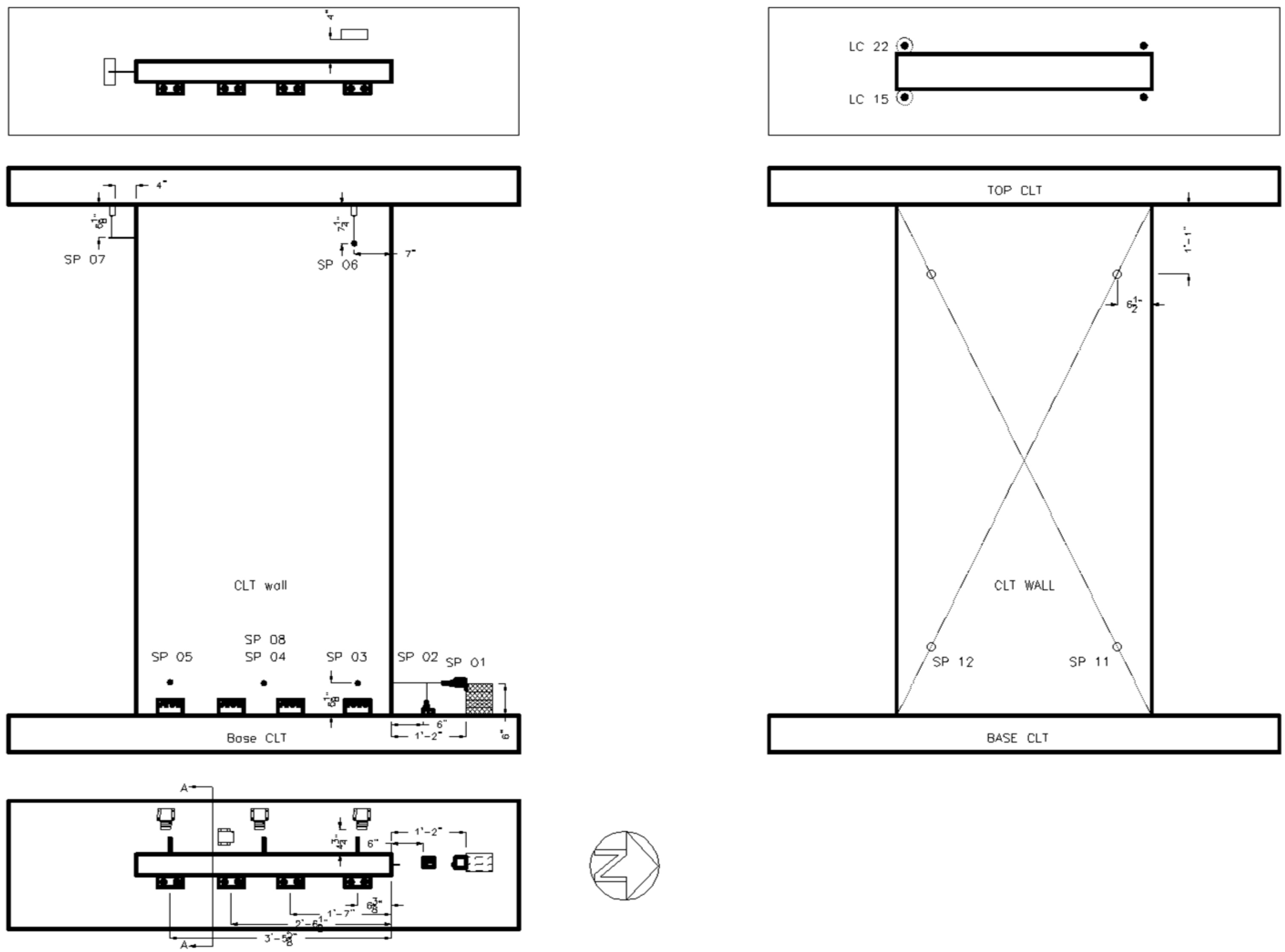


Figure 4.27: Single panel configuration instrumentation

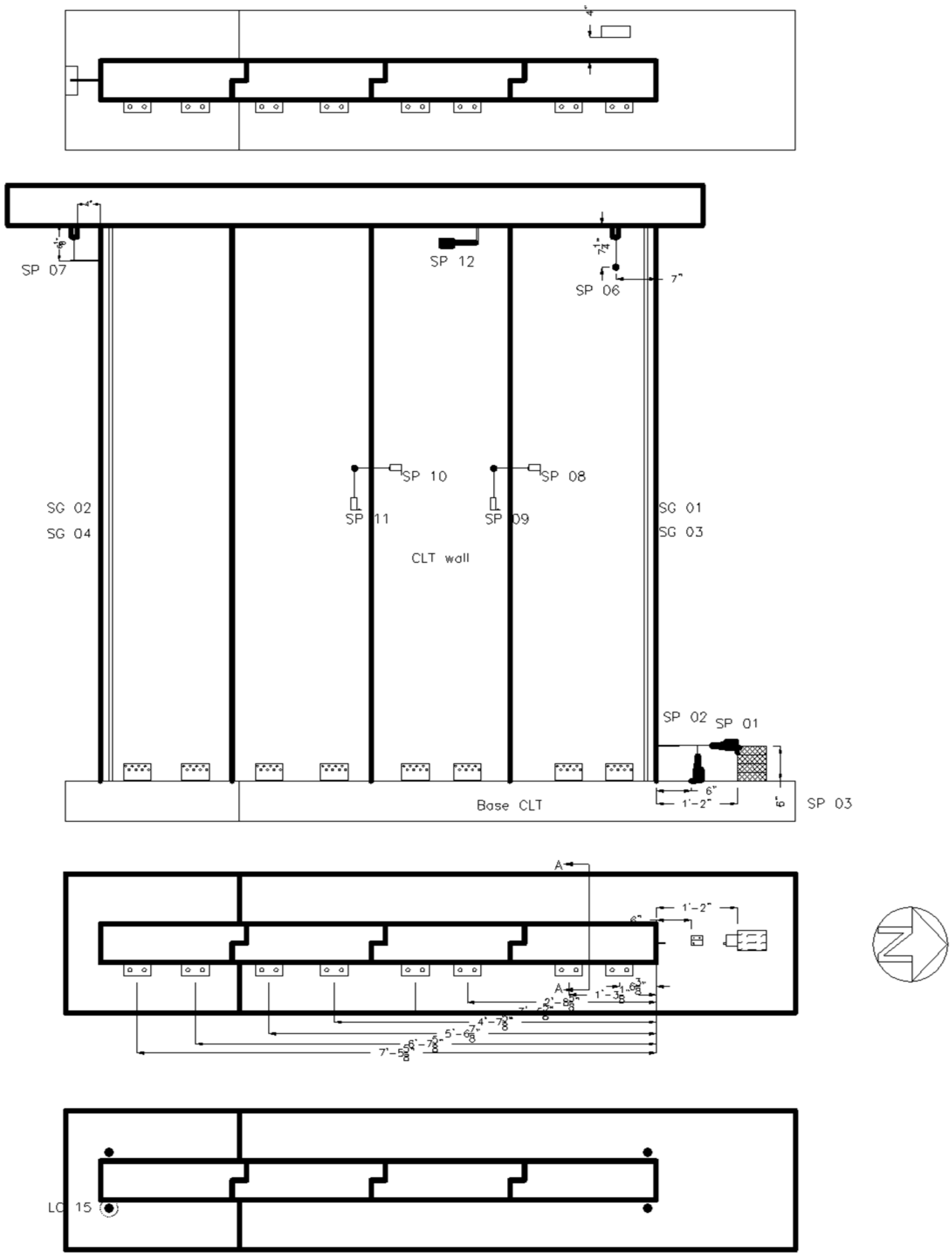


Figure 4.28: Multi-panel configuration instrumentation

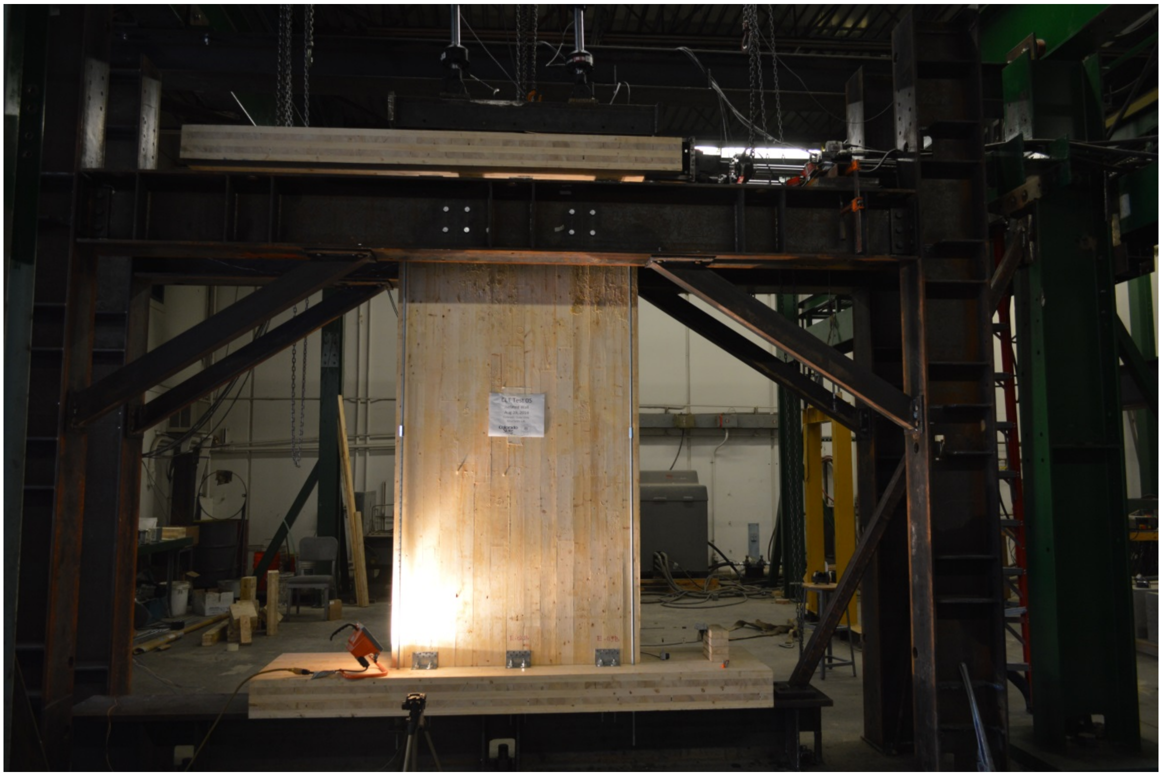
4.2.3 Test results discussion

4.2.3.1 Failure Mechanisms

All the testing configurations with connectors A3 performed well under cyclic loading. Failure mechanisms observed in Test 01 and 02 conducted with A1 type angle bracket did not meet the expected behavior 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 the inter-panel connectors (connectors along the vertical joint) also showed nail yielding and withdrawal. Photos of Test 05 are shown in Figure 4.29 for the purpose of discussion. As seen in the photos the damage is concentrated in the base connectors and nail yielding and withdrawal is observed. One can see the effect of panel rocking on the fasteners by comparing the middle connector to the two edge connectors. The nail withdrawal was significantly greater for the edge connectors. CLT wood crushing perpendicular to the gain at the bottom toe of the wall and the top corner as a result of rocking is also observed.

For the 2:1 aspect ratio panels, the damage was due to a combination of sliding and rocking while in the case of 4:1 aspect ratio panels the damage was mainly due to the rocking. The effect of sliding on hysteresis is evident by looking at hysteresis provided in the Appendix C for Test 03 and 14 in Figures C.3 and C.11, respectively. The corresponding edge connectors before failure are shown in Figure 4.30 and 4.31. It is important to note that both tests were stopped when complete nail withdrawal was observed in the base connectors and while both were 2:1 aspect ratio panels, Test 03 experienced more sliding. Since nail withdrawal due to sliding was sudden as opposed to gradual withdrawal due to rocking, this led to rapid loss in strength observed in the hysteresis.



(a) Test 05, (3)A3 connectors

(b) left connector

(c) middle connector

(d) right connector

(e) top corner of the CLT panel

(f) bottom corner of the CLT panel

Figure 4.29: Test 05

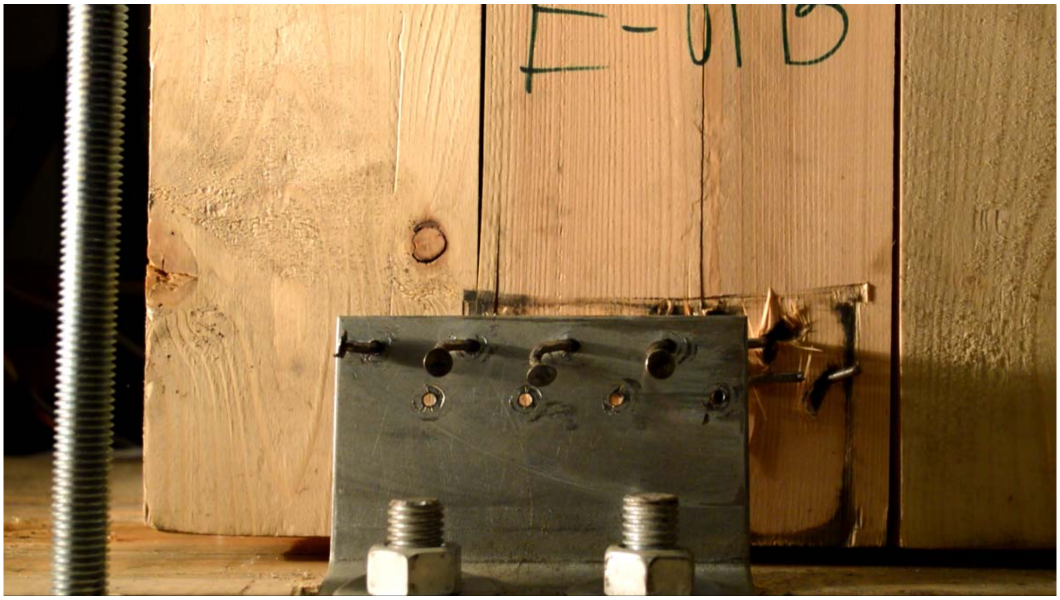


Figure 4.30: Test 03, (3)A3 connectors, edge connector

Figure 4.31: Test 14, (3)A3 connectors, edge connector

As mentioned earlier, in the case of 4:1 aspect ratio panels (Tests 21, 23 and 26), the lateral displacement was primarily due to rocking. This is shown in Figures 4.32-4.34.

(a) CLT panel rocking during the test

(b) edge connector during rocking

Figure 4.32: Test 21, 4:1 aspect ratio panel

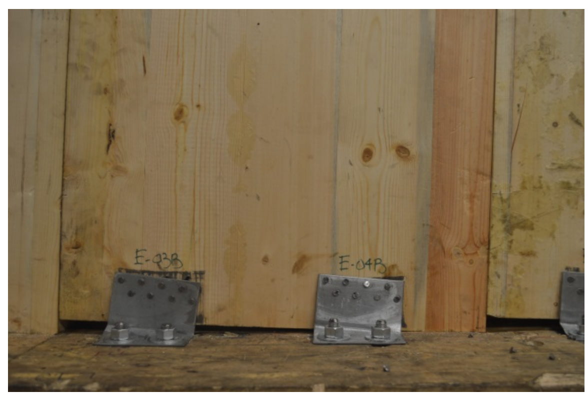
(a) multi-panel configuration rocking during the test

(b) edge connector and inter-panel connector

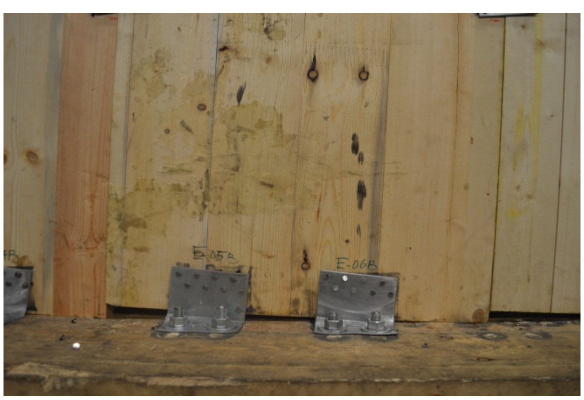
Figure 4.33: Test 23, (2) 2ft x 8ft panel

(a) multi-panel configuration rocking during the test

(b) first panel connectors



(c) second panel connector

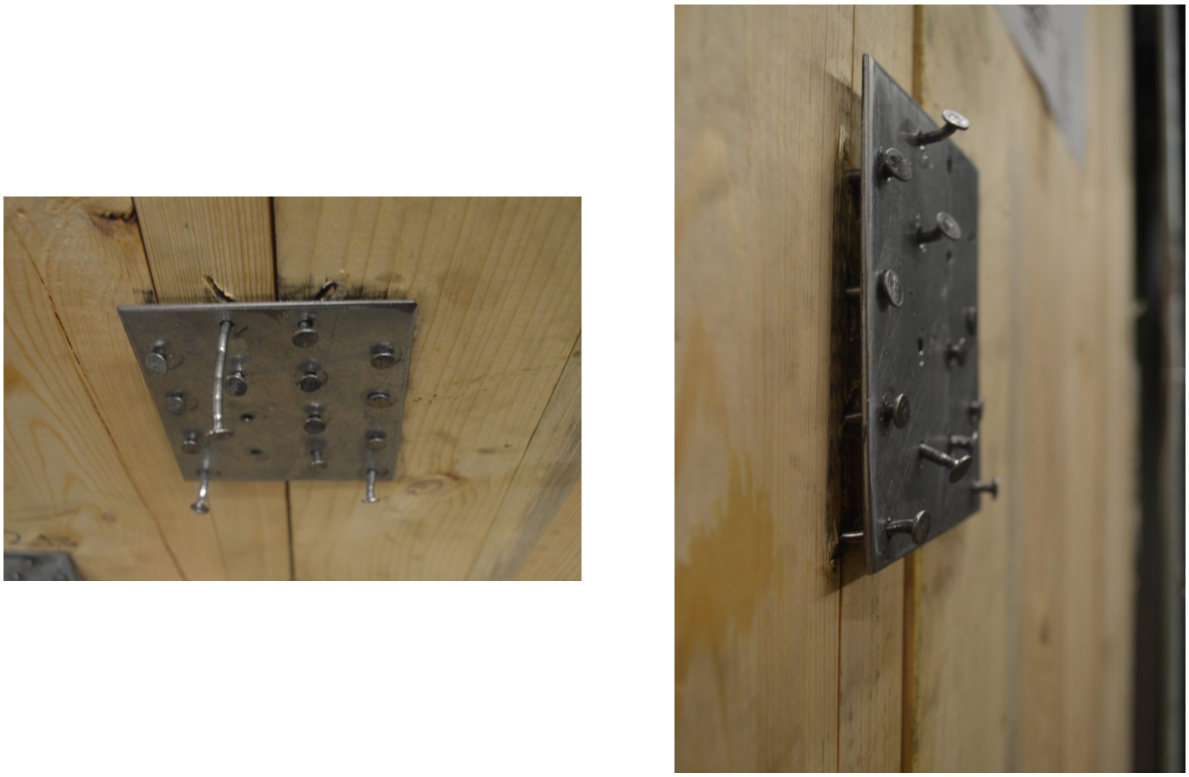


(d) third panel connector

(e) fourth panel connector

Figure 4.34: Test 26, (4) 2ft x 8ft panel

For multi-panel configurations, Tests 23 and 26, the rocking behavior resulted in withdrawal followed by shear failure of some nails in the inter-panel connectors which are shown in Figure 4.35.



(a) Test 23 interpanel connectors

(b) Test 26 inter-panel connectors

Figure 4.35: Inter-panel connectors

4.2.3.2 Connector Thickness

Test 01 and 02 were conducted using the A1 type angle bracket with the dimensions similar to A3 type connector and a thickness of 3/8 in. 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. While 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 4.36. Therefore, the remaining tests were conducted using A3 type connectors. Also, comparing the results obtained from Test 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.

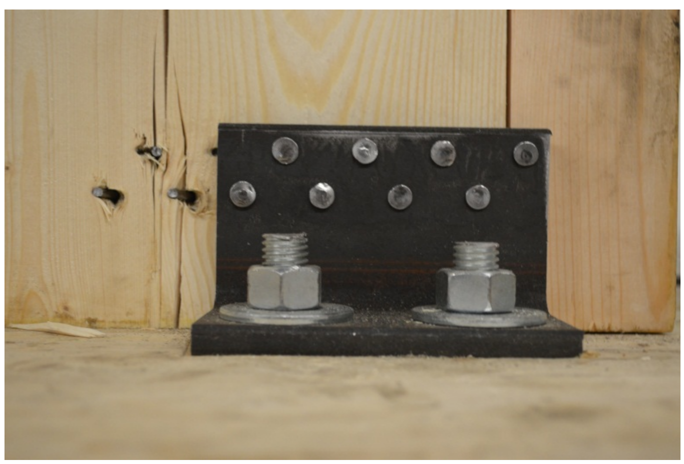


Figure 4.36: A1 type connector after test

4.2.3.3 Boundary Conditions

Based on the FEMA P695 report (FEMA, 2009) 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 since the diaphragm in a structure may be larger compared to 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 effects the rocking of the CLT wall. In order to quantify the effect of a top boundary condition, modifications were made to the original test setup, shown in Figure 4.25, 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 4.37, 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 consider it in post processing the results and developing the corresponding hysteresis.

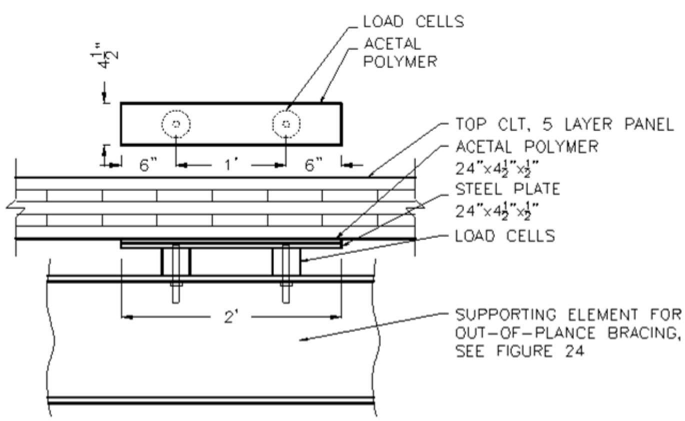


Figure 4.37: Floor diaphragm support

In order 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 a friction coefficient. Friction test results are given in Table 4.14. Once the results were obtained for the friction tests, two specific tests, 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 conditions, while 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 4.38 and the hysteresis for both of these tests is provided in Figure 4.39. From inspection of the hysteresis plots, it was found that the test without the boundary condition imposed produced similar load deformation response with only slight differences in strength, stiffness, and displacement capacity. As a result, additional testing utilized the less complex test set-up without the boundary condition imposed.

Table 4.14. Friction test results

Test No.	Friction coefficient
01	0.330
02	0.281
03	0.280
04	0.210
05	0.387
06	0.317
07	0.337
08	0.289
09	0.292
10	0.271
Average	0.299
COV	0.157

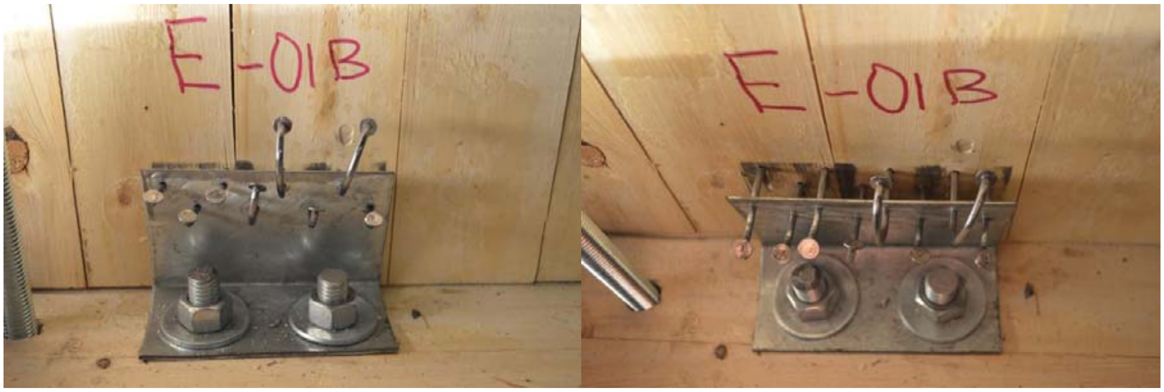


Figure 4.38: Connector failure CLT shear wall Test 05

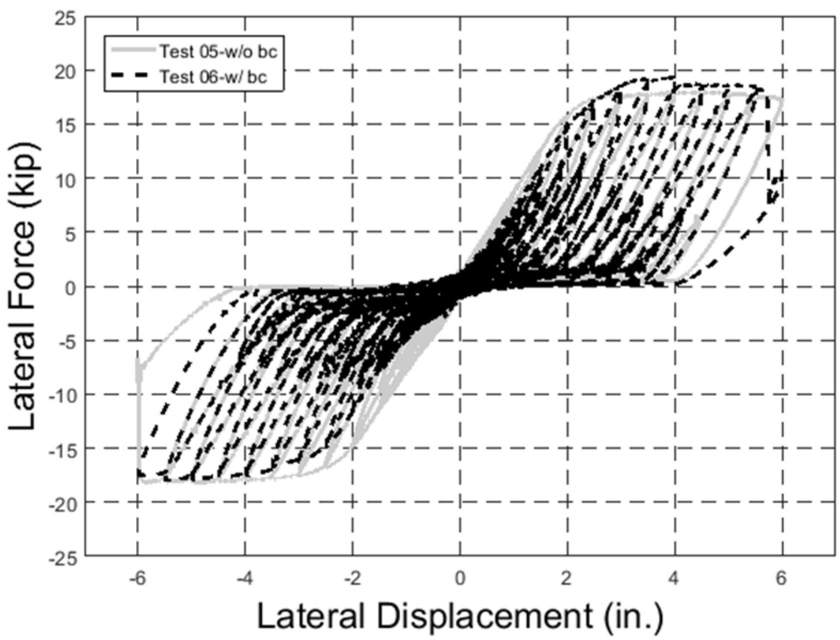


Figure 4.39: Hysteresis with and without boundary condition imposed

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 4ft x 8ft CLT wall under three levels of vertical loads that include no gravity, 0.68 kip/ft, and 1.28 kip/ft were tested and the results for all three tests are shown in Figure 4.40. These tests were Tests are 09, 03, and 04, respectively, in Table 4.10. From Figure 4.40 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.

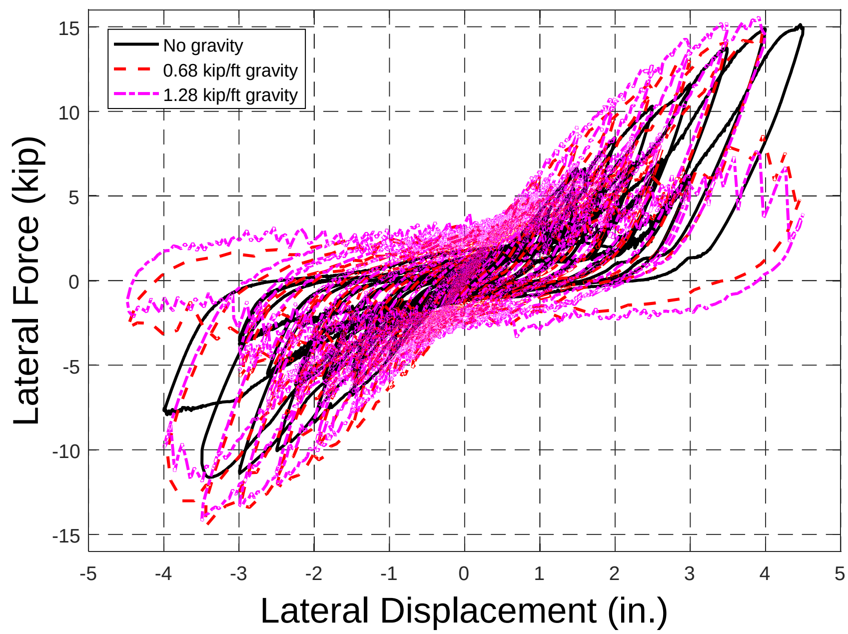


Figure 4.40: 4ft x 8ft x 6.65 in. panel tested under different vertical loading (data smoothing was performed)

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 Figure 4.41 and Figure 4.42. Based on the hysteresis, as one would expect, CLT grade has an influence on strength and stiffness of the CLT panels when the exact same connectors and fasteners are used. A similar trend is observed by comparing Tests 11 and 15, shown in Figure 4.43. The strength of wood connections is significantly influenced by its specific gravity; the higher the specific gravity the denser the wood resulting in higher 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 (2014). E1 grade was found to have on average a higher specific gravity than the

V2 grade. Specified SG 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 layer and inner layer, respectively. Results of the specific gravity tests are provided in Figure 4.44.

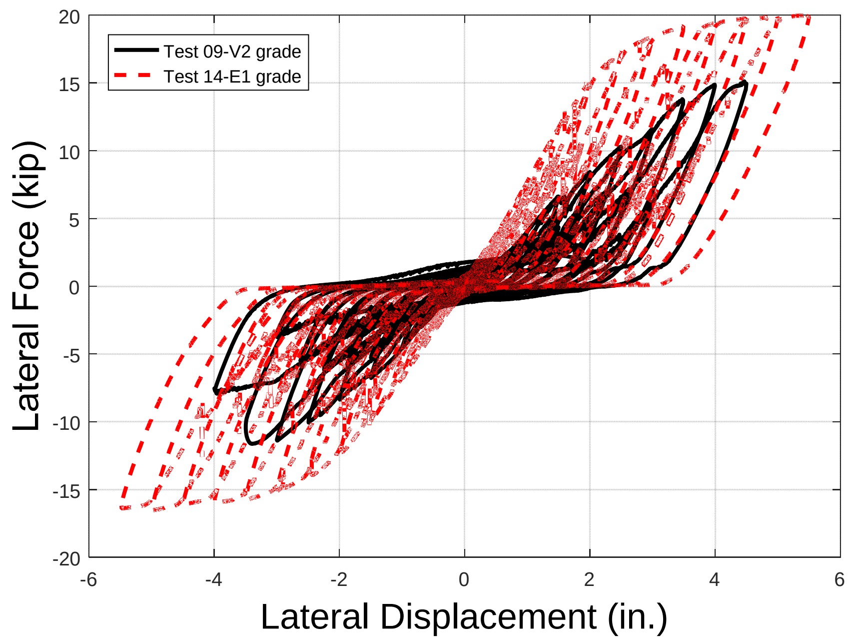


Figure 4.41: Hysteresis on tests on two different grades of CLT

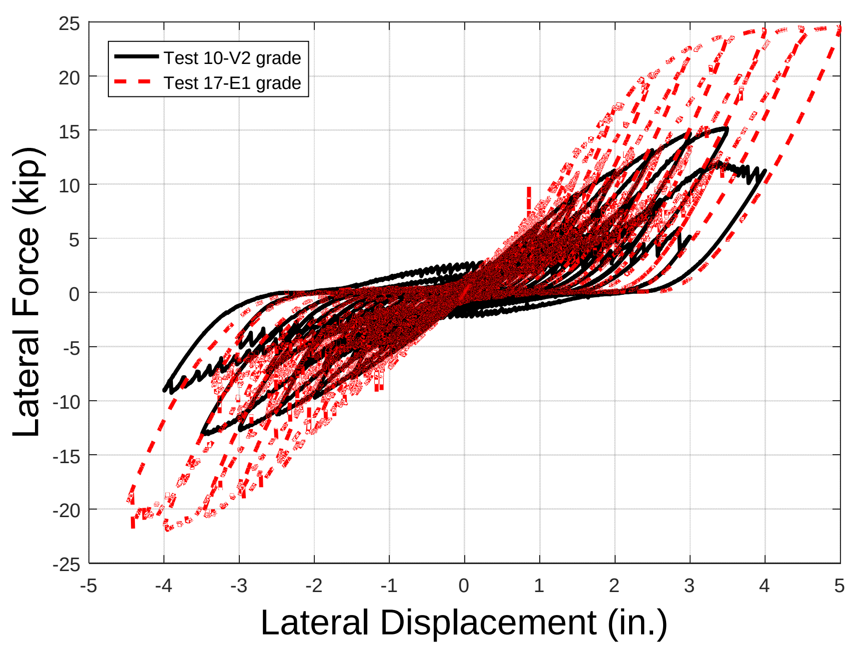


Figure 4.42: Hysteresis on tests on two different grades and thickness of CLT

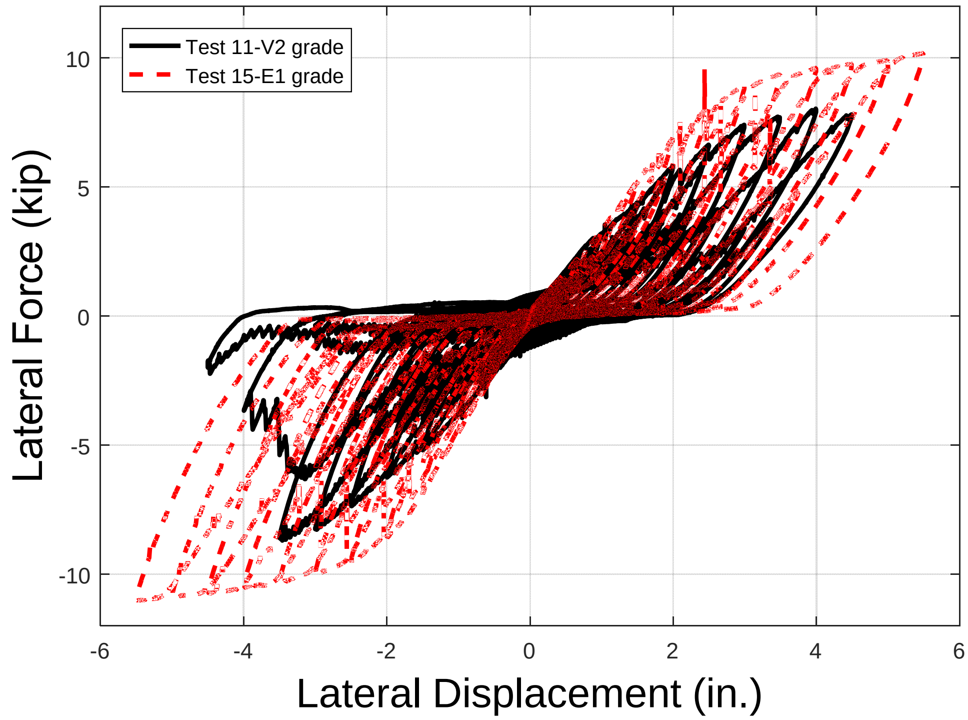
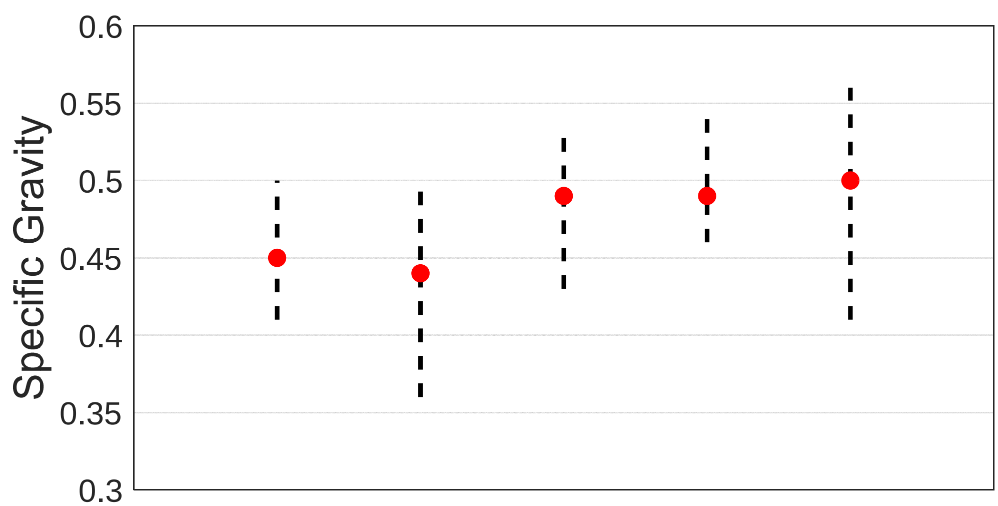


Figure 4.43: Hysteresis on tests on two different grades of CLT



	V2-L-O	V2-X	E1-L-O	E1-X	E1-L1
Max.	0.50	0.50	0.53	0.54	0.56
Min.	0.41	0.36	0.43	0.46	0.41
Avg.	0.45	0.44	0.49	0.49	0.50

Figure 4.44: Specific gravity test results

4.2.3.6 CLT Panel Thickness

Tests 19 and 20 were performed to examine the effect of panel thickness on overall wall behavior. Since CLT is a rocking system, the effect of compression perpendicular to the grain is thought to have an effect on the rocking behavior. Figure 4.45 indicates that there is only a slight difference in the initial stiffness and maximum strength of different thickness panels with the thicker panel being stronger and stiffer of the two. A similar trend was observed by comparing the results of Tests 11 and 18, shown in Figure 4.46; albeit in this case the difference was less significant.

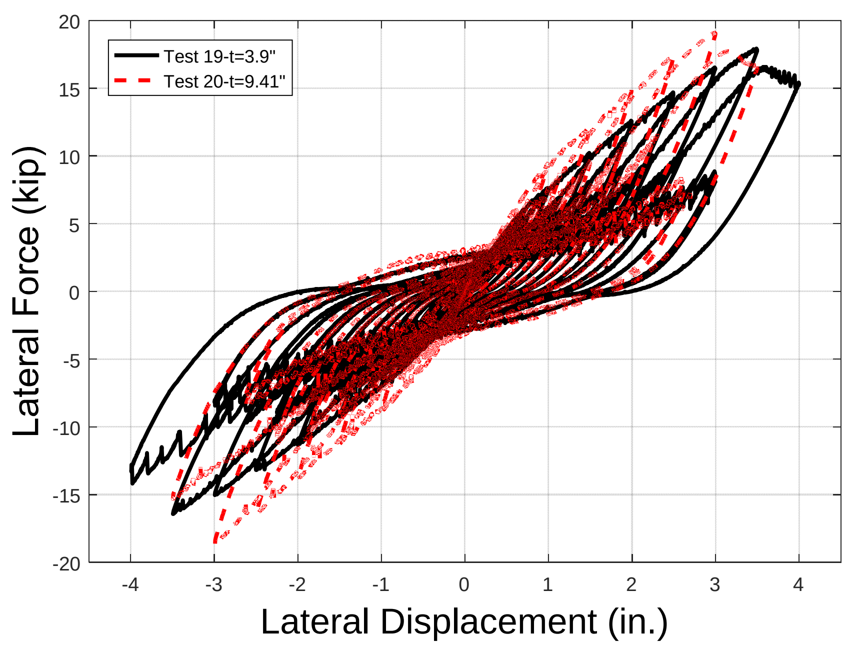


Figure 4.45: Hysteresis for different panel thickness

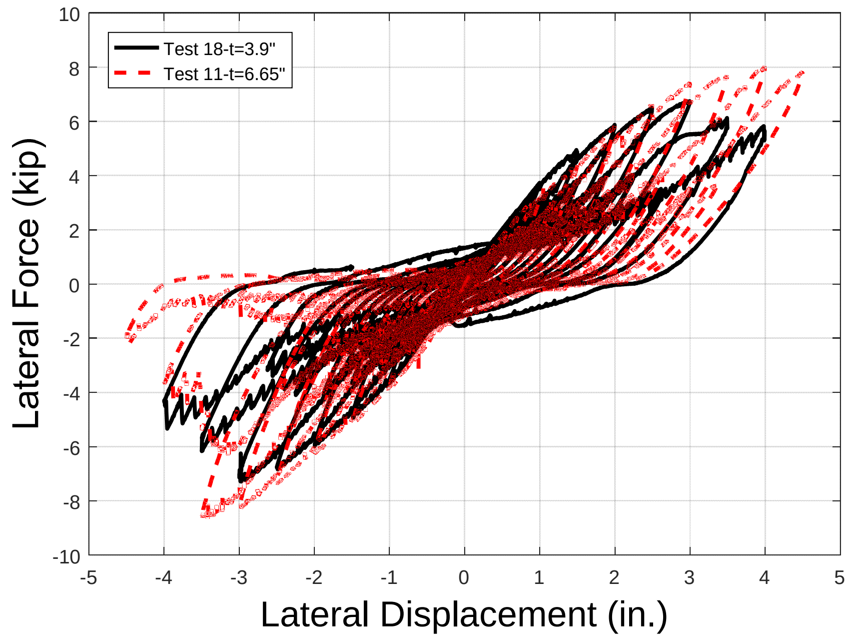


Figure 4.46: Hysteresis for different panel thickness

4.2.3.7 Panel Aspect Ratio

In order to determine the effect of panel aspect ratio, Tests 18 and 21 were performed and the hysteresis compared in Figure 4.47. Results indicate that while higher aspect ratio panel (4:1) exhibited less stiffness and somewhat smaller strength, it had more deformation capacity than the lower aspect ratio panel (2:1) and pinched significantly. This added deformation capacity can be attributed to the rocking behavior of the panel as opposed to rocking and sliding mechanism of other panels tested. However, comparing Tests 22 and 10 in Figure 4.48, the difference between the hysteretic response of a 1:1 and 2:1 panel aspect ratio is minimal. This indicates that there is a lower bound on aspect ratio where it has an insignificant effect on the overall shear wall load-deflection behavior. This can be attributed to the predominately rocking, a combination of rocking and sliding, and sliding behavior of the wall resulting in deformation which correspond to the aspect ratio of the panels that are 4:1, 2:1, and 1:1, respectively.

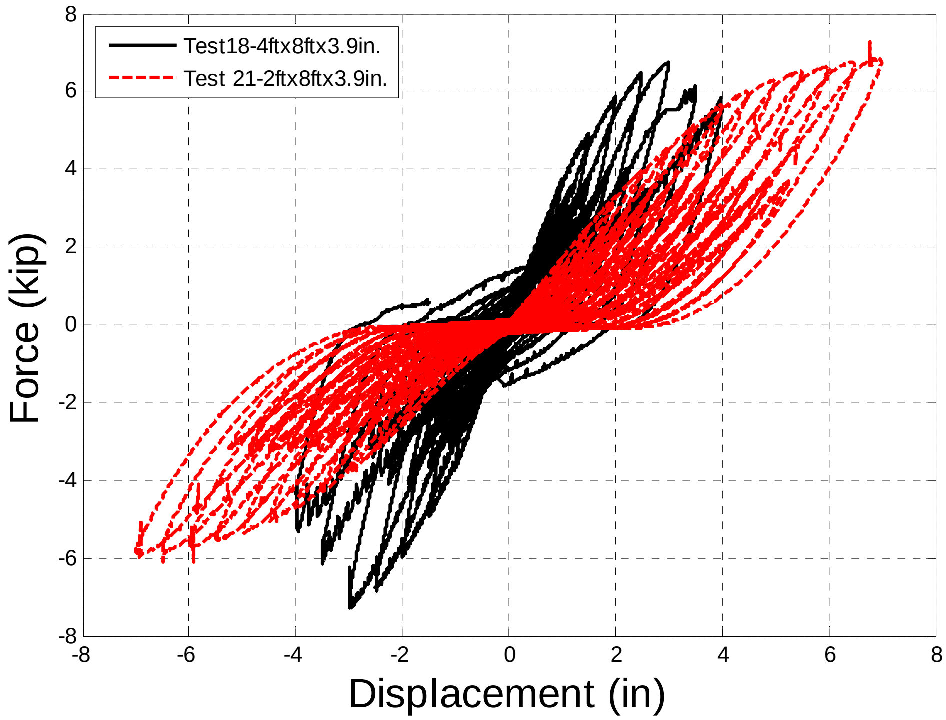


Figure 4.47: Hysteresis for 4:1 and 2:1 panel aspect ratio

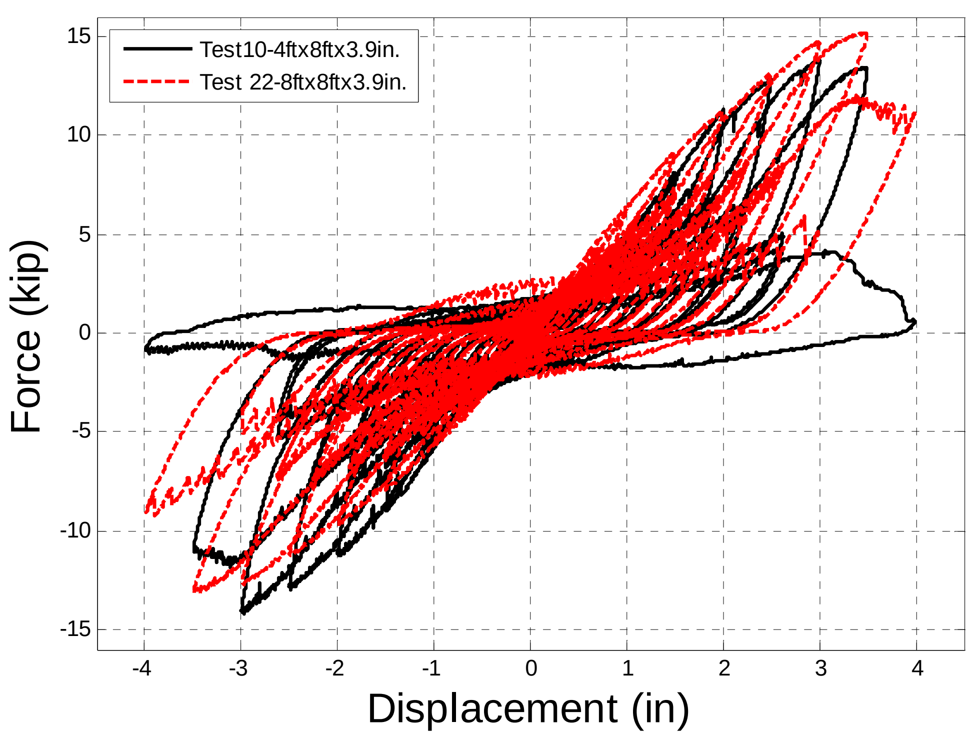


Figure 4.48: Hysteresis for 2:1 and 1:1 panel aspect ratio

4.2.3.8 Inter-panel Connectors

The influence of inter-panel connectors was examined by comparing Tests 23 and 10, although panel thicknesses are different in these two tests it was shown previously that this is not significant. As seen in Figure 4.49, inter-panel connectors add to the deformation capacity of the wall comprised of higher aspect ratio panels, but remain very close in peak capacity with only a slight reduction. This is likely due to reduced stiffness of the inter-panel connectors relative to the holddowns which leads to more uplift demand on the base connectors as the panel rocks. Connectors for the vertical joints provide equivalent shear capacity to that of the angle brackets used in the base and top of the wall, A3 type connectors in this case. Use of alternatives such as LVL or half-lap joints are permissible under the methodology if equivalence is demonstrated through application of the FEMA P795 (2011) methodology. The vertical joint is designed to yield before the shear capacity of the base connectors are reached resulting in rocking of the individual panels and this rocking behavior is intended as part of the CLT shear design method. This behavior was observed in Tests 23 and 26 and is shown in Figures 4.50 and 4.51, respectively. The hysteresis for Test 26 is shown in Figure 4.52 and as seen vertical joints add to the deformation capacity of the wall.

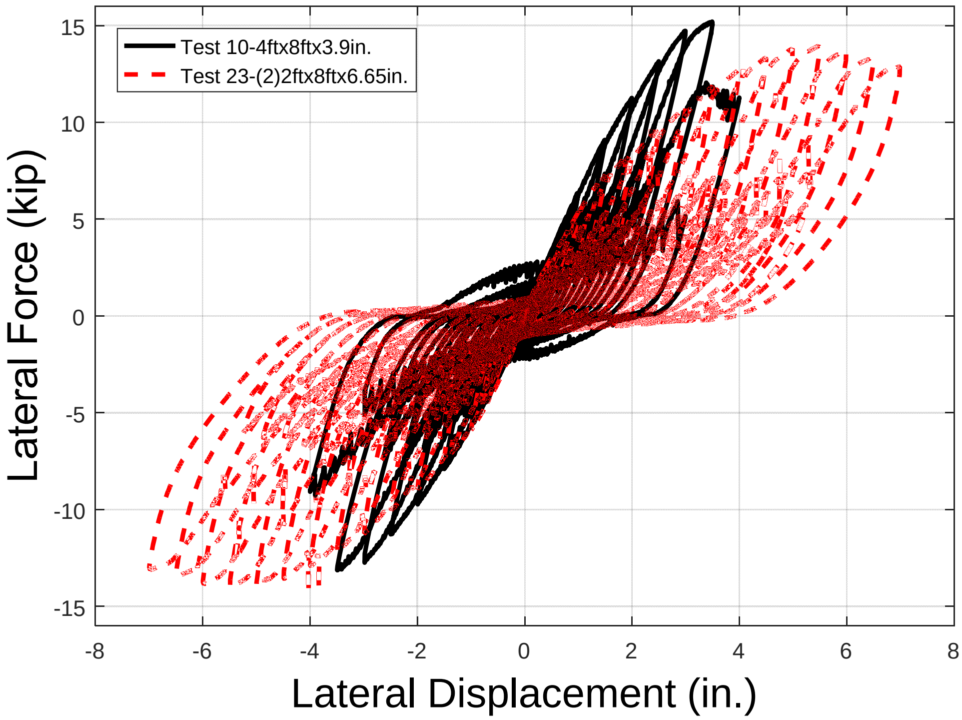


Figure 4.49: Hysteresis for cases with and without inter-panel connector

Figure 4.50: Test 23, shear wall with (2) 2ft panels with vertical joint

Figure 4.51: Test 26, shear wall with (4) 2ft panels with vertical joint

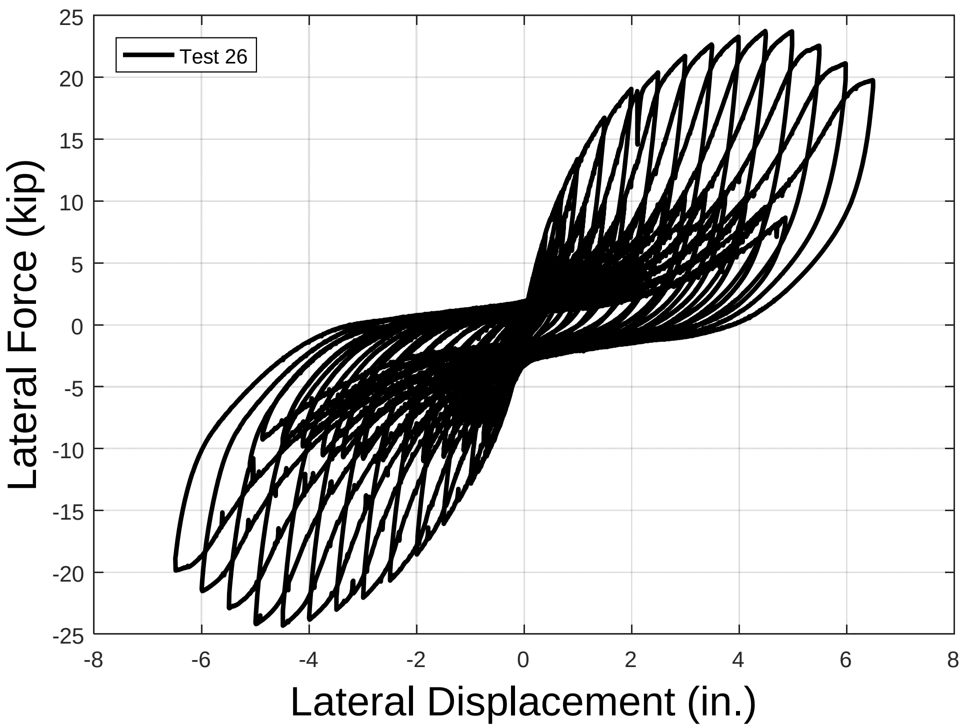


Figure 4.52: Test 26 hysteresis, shear wall with (4) 2ft x 8 ft x 6.65 in. panels with vertical joint

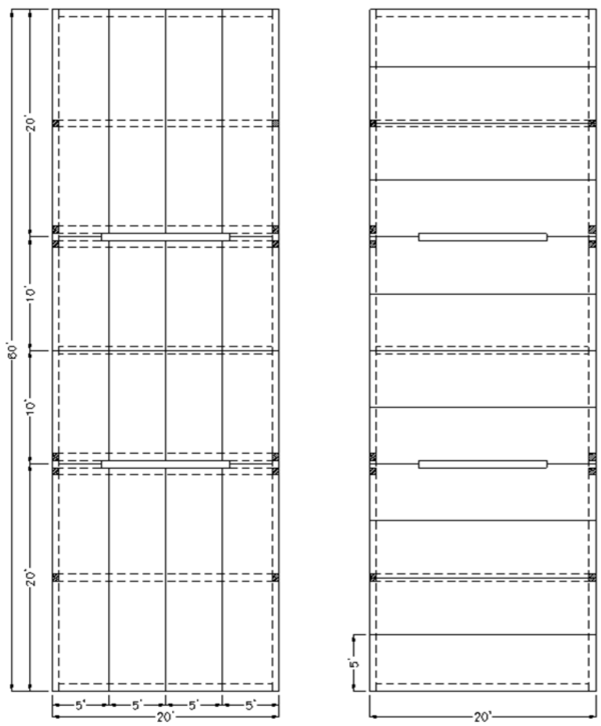
4.3 Shake Table Testing

The purpose of this phase of testing was to investigate the seismic behavior of the CLT shear wall systems in a platform type application. This was a collaborative effort between Colorado State University, Colorado School of Mines, Oregon State University, University of Washington, Washington State University, and a number of partners from the industry. The shake table testing was divided into three phases with each phase consisting of a different seismic force resisting system (SFRS) all based on CLT. Phase I and II were a rocking based resilient system while Phase III which also corresponds to Phase 3 of the testing discussed in this dissertation, was designed based on the methodology developed here.

The structure was considered for a location in San Francisco and the design forces were obtained based on Section 12.8 of ASCE 7-16 for an assumed response modification factor, R, of 4 and Φ=0.55. It is important to note that R and Φ were merely design assumptions at the time and these values are different than the values presented later in the document which are calculated based on the rigorous FEMA P695 procedure. Further assumptions are stated in Section 4.3.3 of this document and the design was performed based on design methodology that is presented in Chapter 6.

4.3.1 Specimen Description

The two-story structure consisted of a 2400 ft^2 diaphragm supported by the gravity system comprised of glulam beams and columns. The floor plans for the structure are shown in Figure 4.53. CLT panels were used for both top and bottom floor in different orientations. The first floor diaphragm was constructed with 5 ft x 20 ft x 4.125 in. CLT panels and 5 ft x 9 ft x 4.125 in. CLT panels in the exterior and interior, respectively. The second floor was constructed with 5 ft x 20 ft x 4.125 in. panels oriented in their longer side and 2.25 in. concrete flooring was added to investigate the behavior of composite CLT flooring. Test setup with the gravity system is shown in Figure 4.54.



(a) (b)

Figure 4.53: Specimen Floor Plans (a) First Floor (b) Second Floor

Figure 4.54: Specimen under construction, only gravity frame is seen in the photo

Shake table testing was performed in three subphases with each phase consisting of different CLT wall configurations. These are referred to henceforth as Phase 3.1, 3.2 and 3.3. Panel configurations for all the phases are shown in Figures 4.55-4.57. Phases 3.1 and 3.2 were multi-panel configurations with 4:1 (panel height/panel width =h/b) and 2:1 aspect ratio panels, respectively. Phase 3.3 is similar to Phase 3.1; however, transverse walls were added in Phase 3.3 to investigate their effect on the behavior of the shear walls and response of the building.

(a) Second Floor

(b) First Floor

Figure 4.55: Phase 3.1 (4) 4:1 aspect ratio panels

(a) Second Floor

(b) First Floor

Figure 4.56: Phase 3.2 (2) 2:1 aspect ratio panels

(a) Second Floor

(b) First Floor

Figure 4.57: Phase 3.1 (4) 4:1 aspect ratio panels with perpendicular walls

4.3.2 Instrumentation

A general instrumentation plan that remained unchanged throughout the project was developed to investigate behavior of the diaphragm and the gravity system. Instrumentation types included linear and string potentiometers, strain gauges and accelerometers. The instrumentation type and placement was intended to study response at the global and at the component levels. This included drift and acceleration at each story, relative vertical movement of each story, relative movement of the floor panel with respect to each other and the gravity system, and load and relative displacement at the metal chord splices of the diaphragm.

Additional instrumentation was provided to specifically investigate the seismic force resisting system (SFRS) used in the testing. Figures 4.58-4.60 shows instrumentations on CLT walls used in Phase 3.1-3.3. The instrumentation types included string potentiometers, linear spring potentiometer, accelerometers, and load cells. These are provided to measure sliding and rocking of the CLT panels, relative movement of the panels with respect to the base and the floor diaphragm above, slip between the adjacent panels, and loads in the overturning moment restraint.

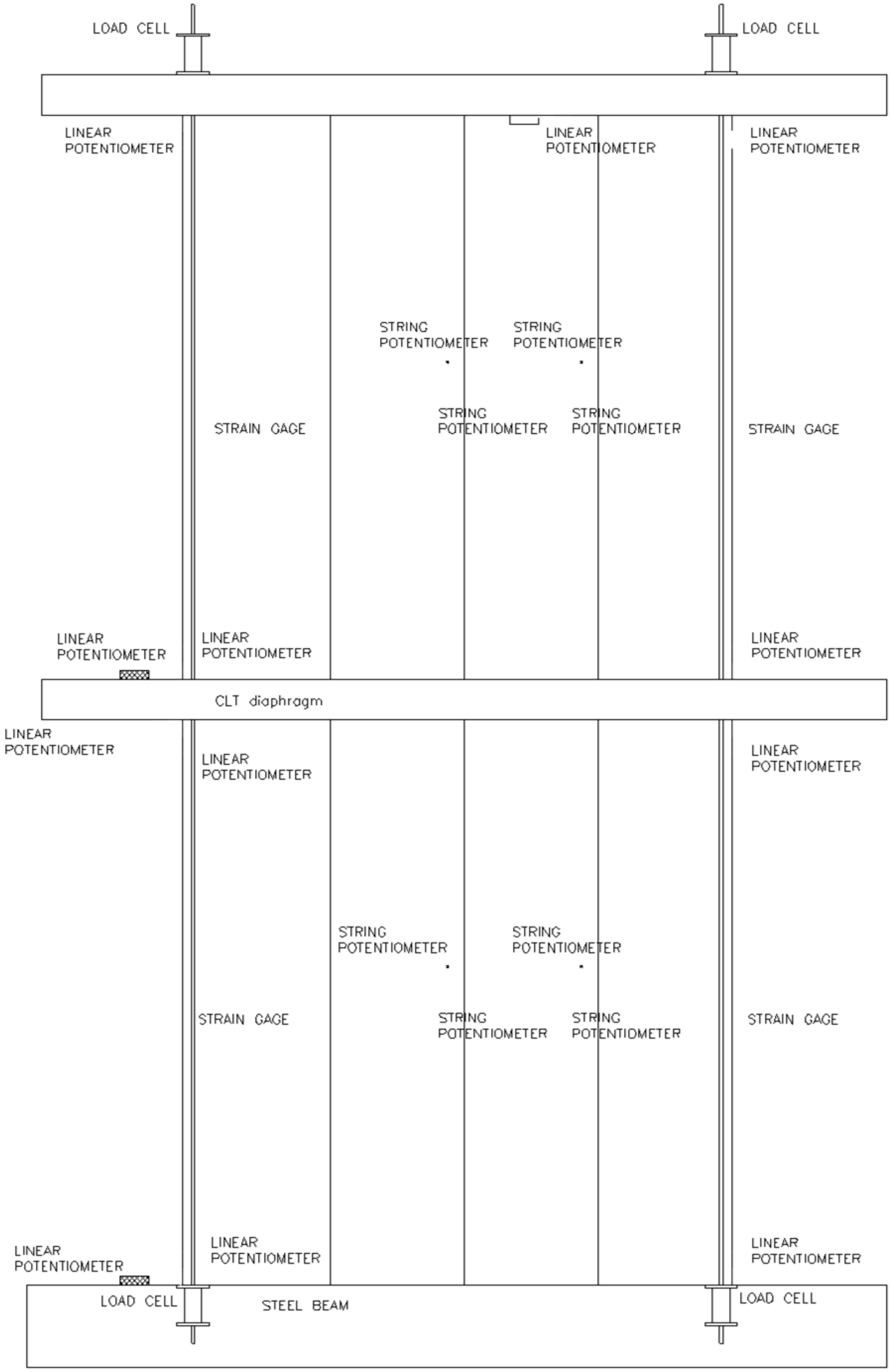


Figure 4.58: Instrumentation on north face of south wall, Test Configuration 1

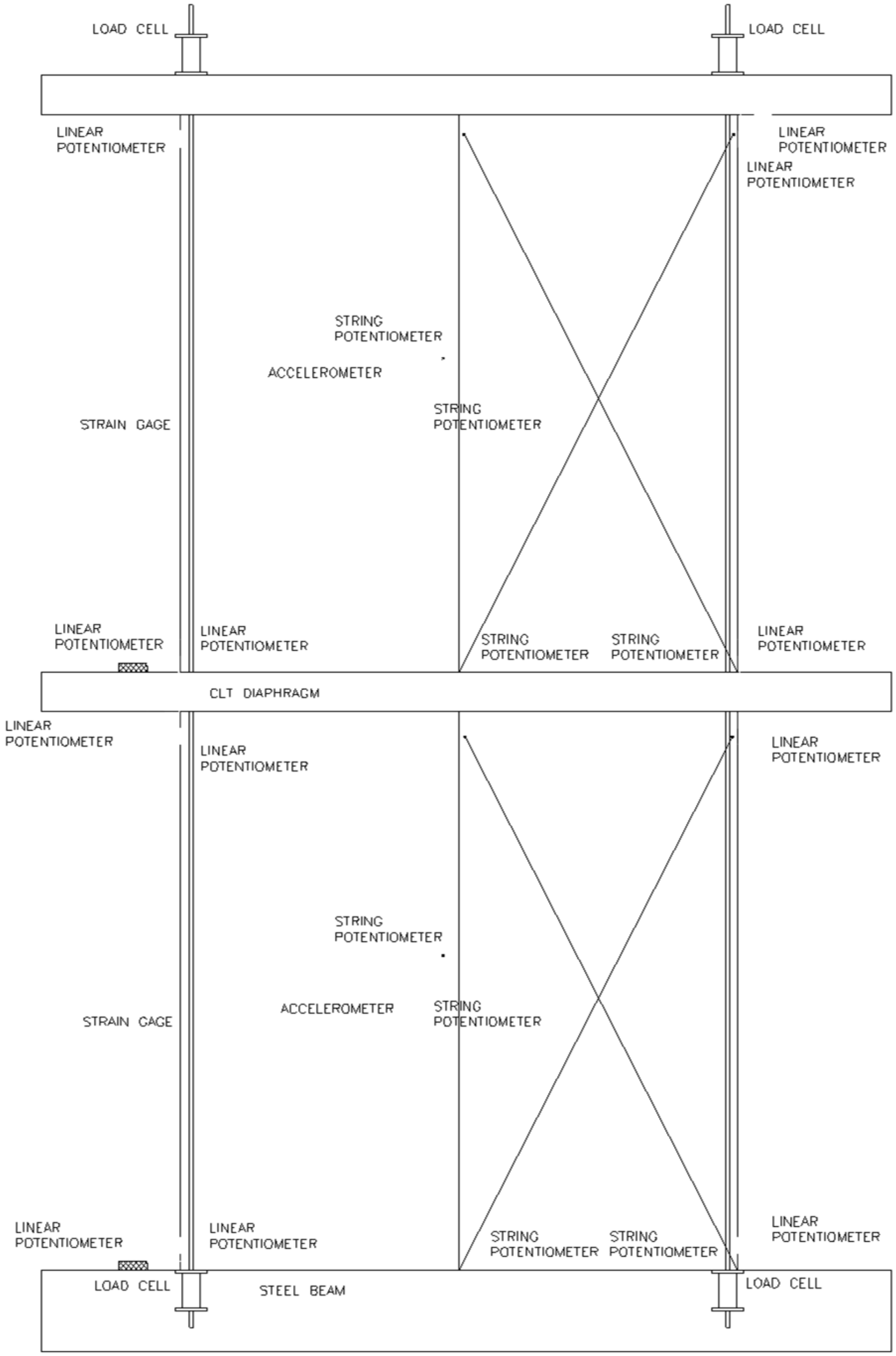


Figure 4.59: Instrumentation on north face of south wall, Test Configuration 2

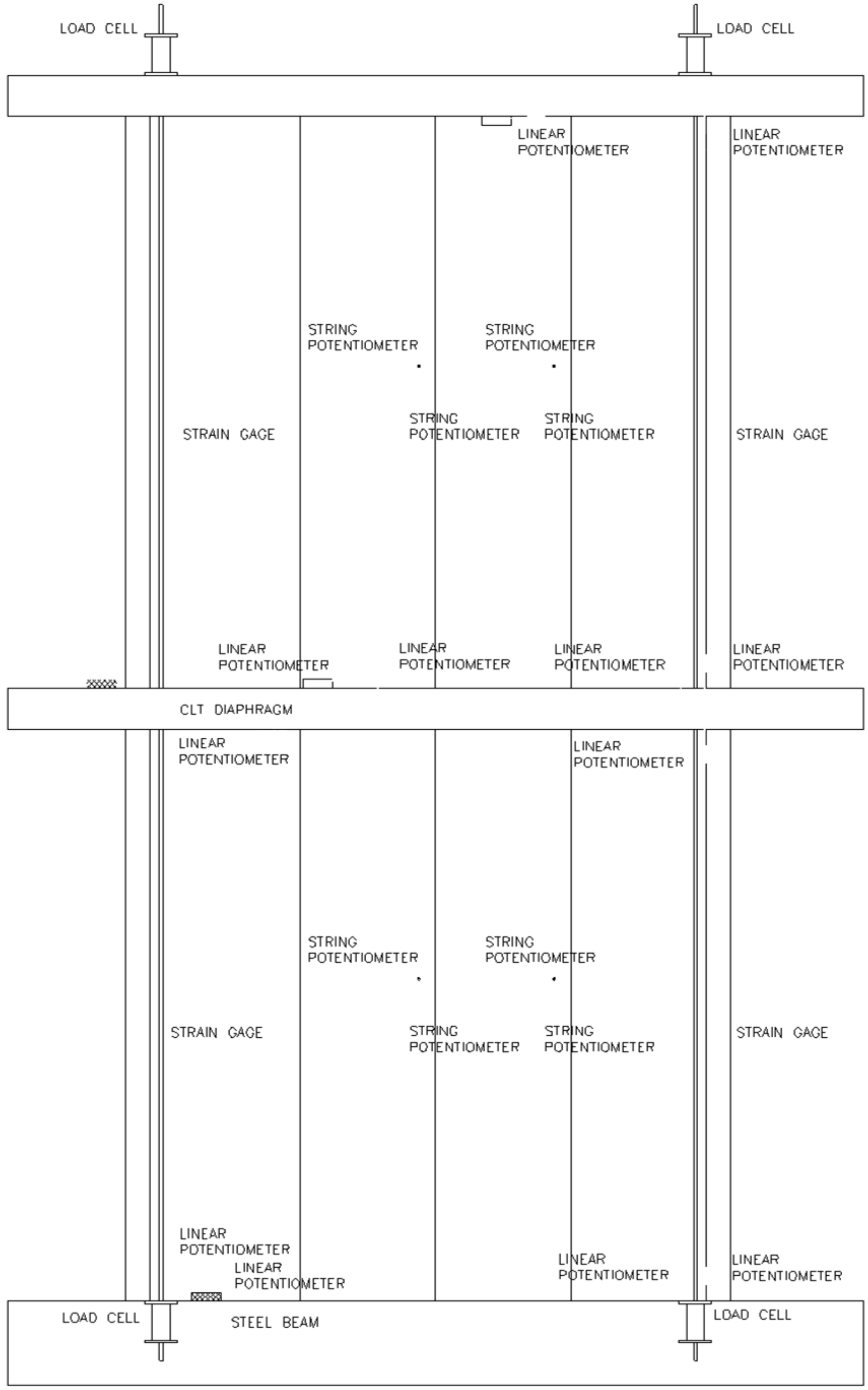


Figure 4.60: Instrumentation on north face of south wall, Test Configuration 3

4.3.3 Ground Motion Scaling

The test building was assumed to be located in San Francisco with the site classification taken as Site Class D and the risk category is I or II. Three levels of seismic hazard were considered that include frequent earthquake, design earthquake, and maximum considered earthquake, each corresponding to a mean return period of 72 years, 474 years, and 2475 years, respectively. The 5%-damped design spectrum parameters for DE (Design Earthquake) and MCE (Maximum Considered Earthquake) obtained based on NEHRP 2015 provisions are shown in Figure 4.61. For the frequent earthquake that corresponds to a mean return period of 72 years, spectral acceleration was calculated based on Section 1.6.1.3 of FEMA 356 (2000). The equation is:

$$\boldsymbol{S_{i,PE} = S_{i,10/50}\left(\frac{P_R}{475}\right)^{n}, for\ P_R \leq 475} \qquad \textbf{(4.1)}$$

where *i*=*S* or *1* referring to short-period and 1-sec spectral values, respectively, P_R is return period, and n=0.44 for California based on Table 1-3 of FEMA 356; $S_{50/50}$ was calculated as 44% DE. These values are provided in Table 4.15.

All the tests were conducted using the 1989 Loma Prieta earthquake record provided as part of the far-field ground motion suite in FEMA P695. Information on the ground motion is presented in Table 4.16 and the record is shown in Figure 4.62. The scaling was performed in accordance with the FEMA P695 methodology with the ground motion scaled to the aforementioned three levels of intensities with the corresponding response spectrum shown in Figure 4.63.

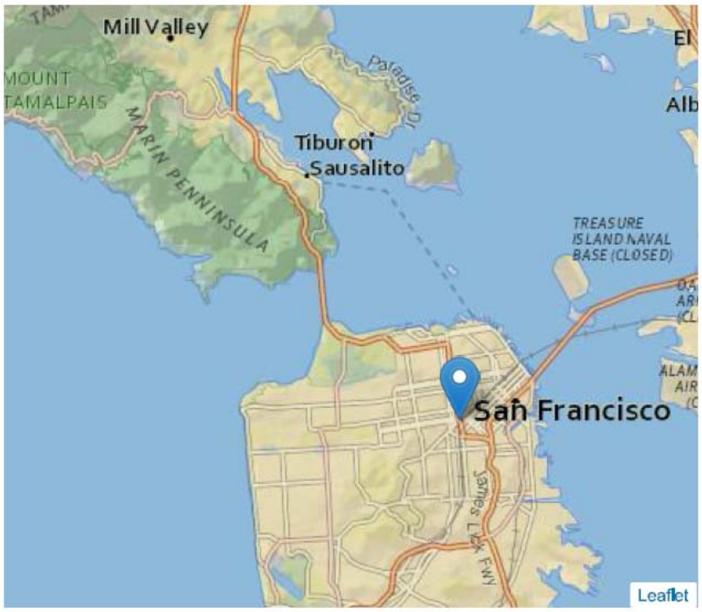
5/10/2017 U.S. Seismic Design Maps

U.S. Geological Survey - Earthquake Hazards Program

UCSD

Latitude = 37.775°N, Longitude = 122.419°W

Location



Reference Document

2015 NEHRP Provisions

Site Class

D (determined): Stiff Soil

Risk Category

I or II or III

S_S =	1.500 g	S_{MS} =	1.500 g	S_{DS} =	1.000 g
S_1 =	0.600 g	S_{M1} =	1.020 g[1]	S_{D1} =	0.680 g[1]

[1] Since the Site Class is D and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

Figure 4.61: Design spectrum parameters for downtown San Francisco

Table 4.15. Design spectral acceleration values for 5% damping

Downtown San Francisco					
Hazard level	Intensity (% of DE)	Exceedance probability	Mean return period (yrs)	Short-period Sxs (g)	1 S S_{X1} (g)
Short return period	44	50%/50yr	72	0.44	0.30
DE	100	10%/50yr	474	1	0.68
MCE	150	2%/50yr	2475	1.5	1.02

Table 4.16. Information on ground motion used in testing

Peer Record Seq. No.	Earthquake	Recording Station	Year	M_w	PGA_{max}(g) recorded
752	Loma Prieta	Capitola	1989	6.9	0.53

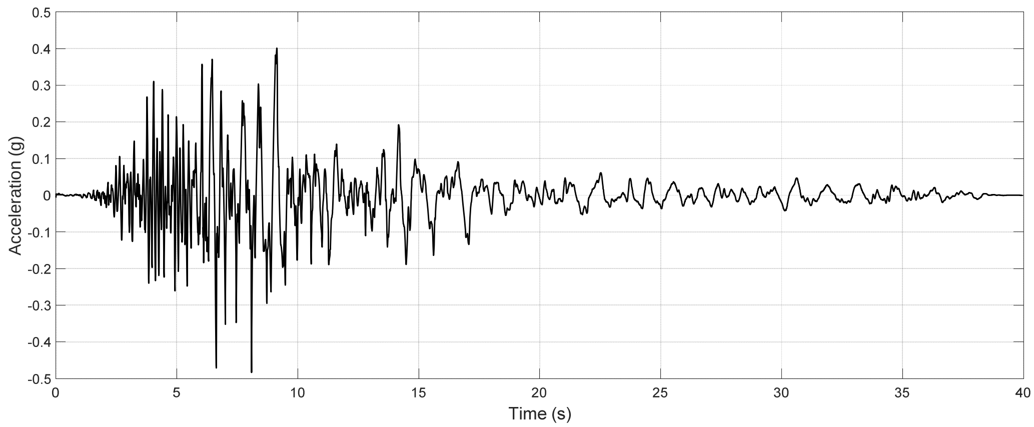


Figure 4.62: Loma Prieta, Capita, Component 2 Normalized

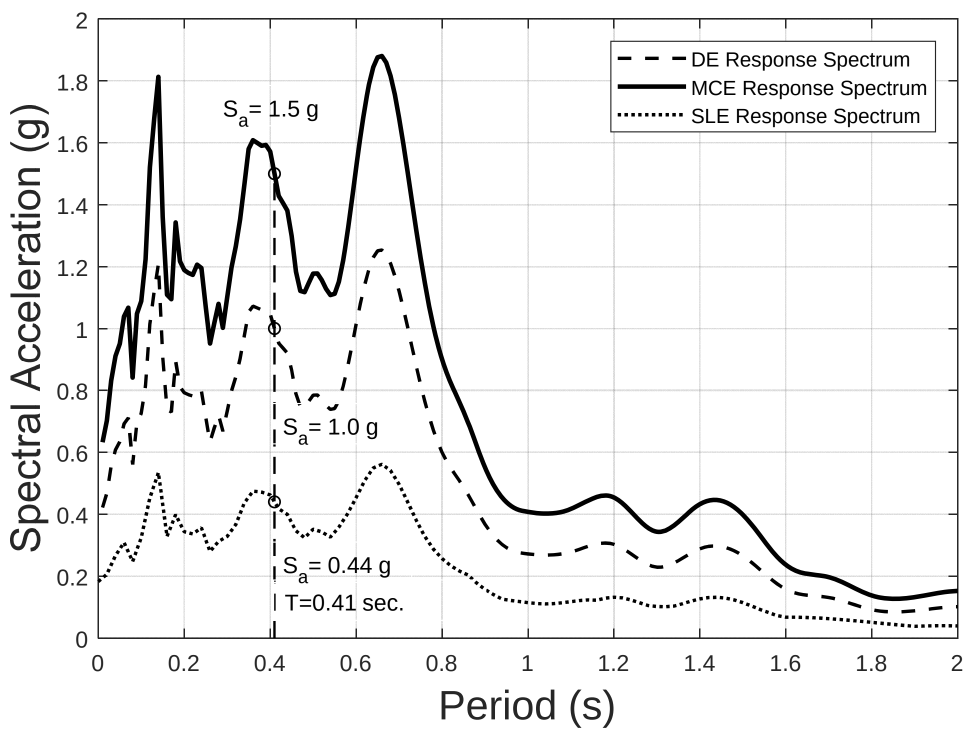


Figure 4.63: Response spectrum scaled to three hazard levels

4.3.4 Test results

In addition to the seismic testing, white noise tests were performed before and after each test to determine natural period of the structure in the loading direction. The testing schedule along with the results of the white noise tests are provided in Table 4.17. A Fourier transformation was used on the accelerometer data to determine frequency content of the data to obtain natural period of the structure. These are shown in Figures 4.64-4.68.

Table 4.17. Shake Table Testing Schedule

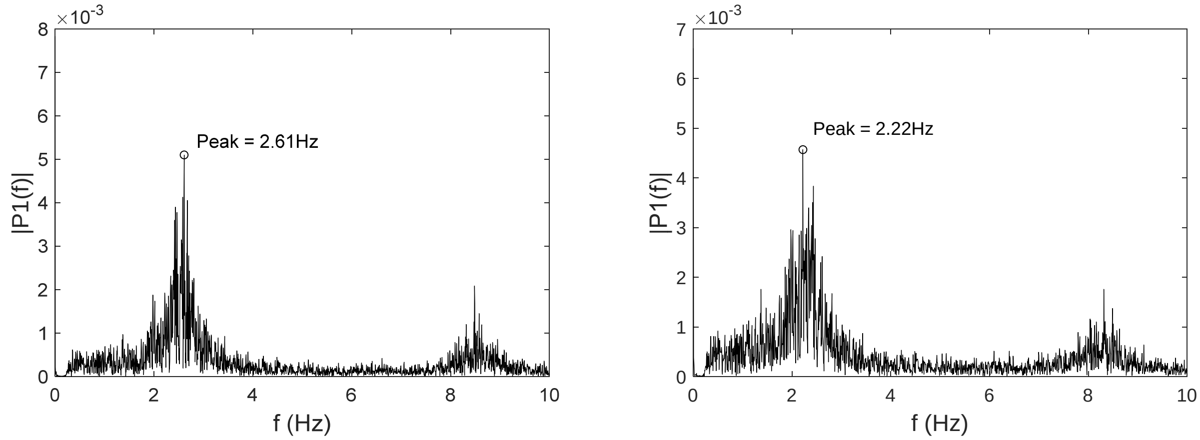
Test Number	Test Configuration	Ground Motion	Freq. (Hz)	Building period (sec)
01	1	WN-01	2.61	0.38
02		Loma Prieta SLE		
03		WN-02	2.22	0.45
		Damage Inspection		
04		WN-03	2.43	0.41
05		Loma Prieta MCE		
06		WN-04	1.37	0.73
		Repair, connectors were replaced		
07		WN-05	2.43	0.41
08		Loma Prieta DE		
09		WN-06	1.98	0.51
10	2	WN-07	2.43	0.41
11		Loma Prieta SLE		
12		WN-08	2.34	0.43
		Damage Inspection		
13		Loma Prieta MCE		
14		WN-09	1.37	0.73
		Damage Inspection		
15	3	WN-10	2.9	0.34
16		Loma Prieta SLE		
17		WN-11	2.61	0.38
		Damage Inspection		
18		Loma Prieta MCE		
19		WN-12	1.37	0.73
		Damage Inspection		

The fundamental period of the structure based on ASCE 7-16 is calculated as shown below. This was presented earlier in Eq. 5 in Chapter 3.

$$T = C_u T_a = C_u C_t h_n^x$$

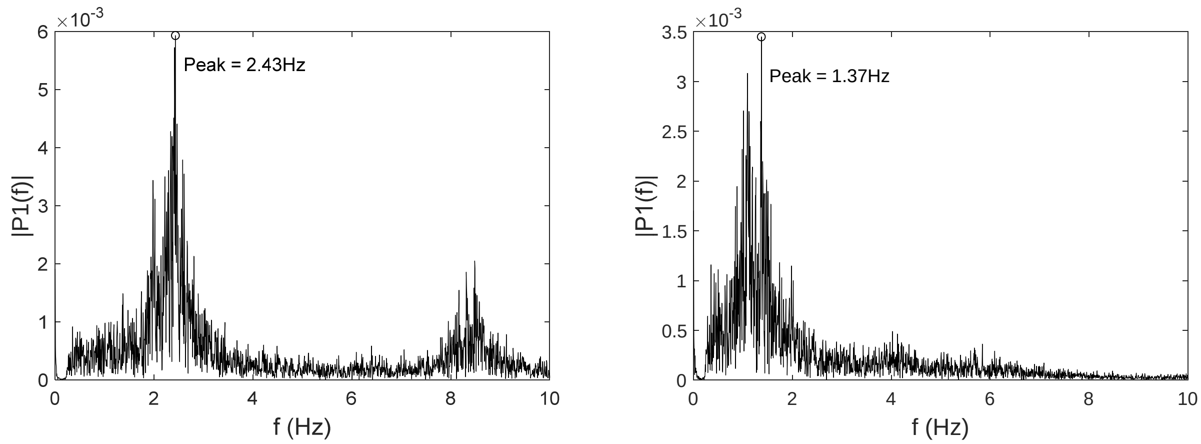
Using h_n= 22ft, C_u=1.40, C_t=0.02 and x=0.75: T=0.284 sec.

Looking at the table, the initial natural period obtained from the white noise tests prior to induced damage (elastic period) are larger than the value obtained through the period formula in ASCE 7 (2016). It important to note that the period formula based on ASCE 7-16, C_t and x parameters were selected for the case of all other structural systems, a case that applies to all the structures excluding steel and concrete moment resisting frames, steel eccentrically braced frames and steel buckling-restrained frames. This signifies the importance of a period formula for CLT structures.



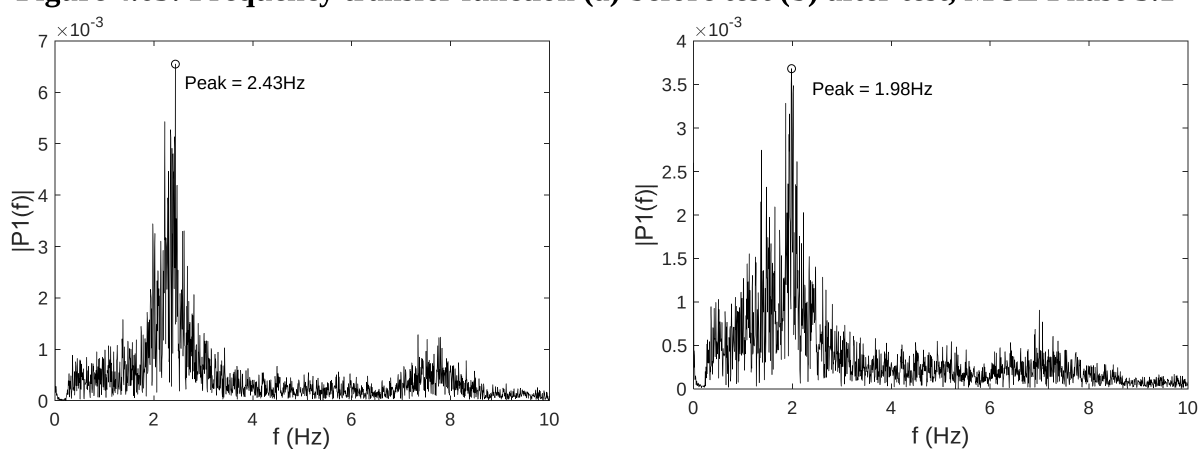
(a) WN-01 **(b) WN-02**

Figure 4.64: Frequency transfer function (a) before test (b) after test, SLE Phase 3.1



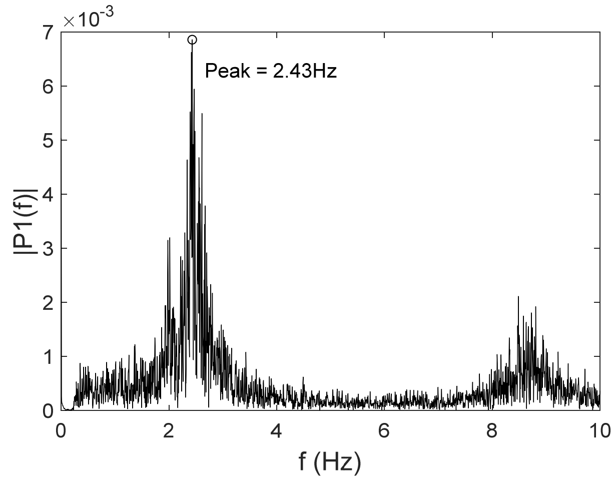
(a) WN-03 **(b) WN-04**

Figure 4.65: Frequency transfer function (a) before test (b) after test, MCE Phase 3.1

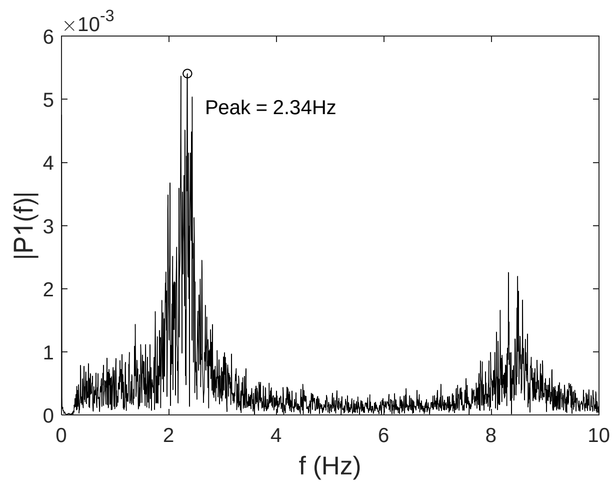


(a) WN-05 **(b) WN-06**

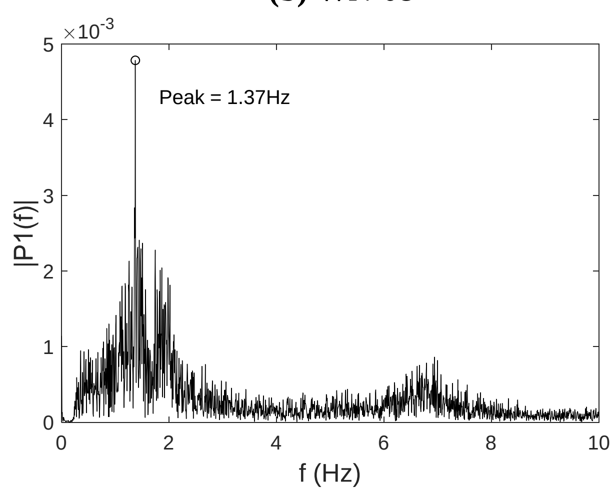
Figure 4.66: Frequency transfer function (a) before test (b) after test, MCE Phase 3.1



(a) WN-07

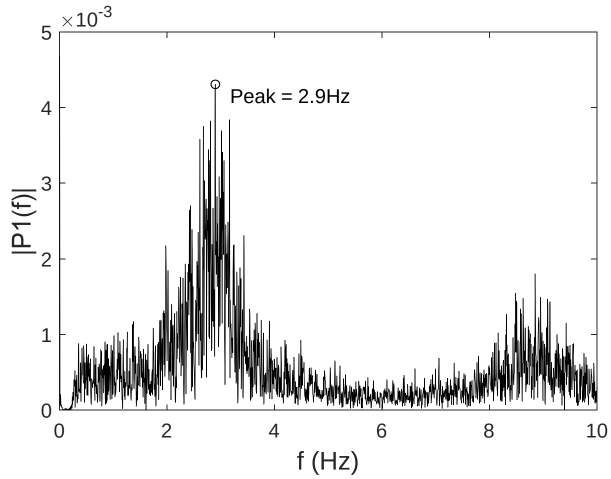


(b) WN-08

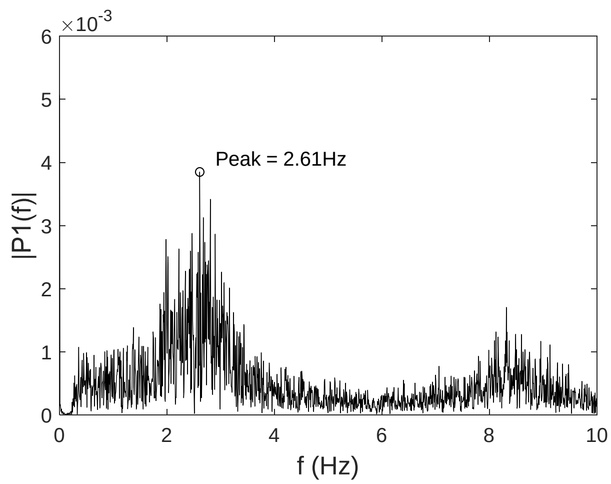


(c) WN-09

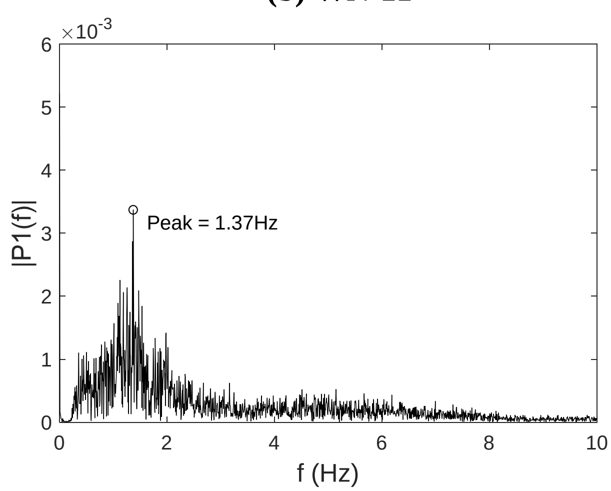
Figure 4.67: Frequency transfer function (a) before test SLE 3.2 (c) after SLE 3.2 and before MCE 3.2 (c) after MCE 3.2



(a) WN-10



(b) WN-11



(c) WN-12

Figure 4.68: Frequency transfer function (a) before test SLE 3.3 (c) after SLE 3.3 and before MCE 3.3 (c) after MCE 3.3

4.3.4.1 Load Deformation Behavior

The deformation shapes of the structure for different phases of testing are shown in Figure 4.69 and as seen in the figure, the inter-story drifts exhibited a linear deformation profile. For the MCE ground motion the maximum displacement at the roof was 6.17 in., 6.16 in. and 5.9 in. for Phases 3.1, 3.2, 3.3, respectively. The configuration with 2:1 aspect ratio panels had the same displacement as the case with 4:1 aspect ratio panels. However, in the latter case a slightly larger portion of the displacement was concentrated in the first story. This was mainly due to rocking of the panels in the bottom floor. The influence of perpendicular wall is noted by comparing the results obtained for Phase 3.1 and 3.3. The overall roof displacement is reduced from 6.16 in. to 5.9 in. Inter-story drifts calculated based on story heights are given in Table 4.18.

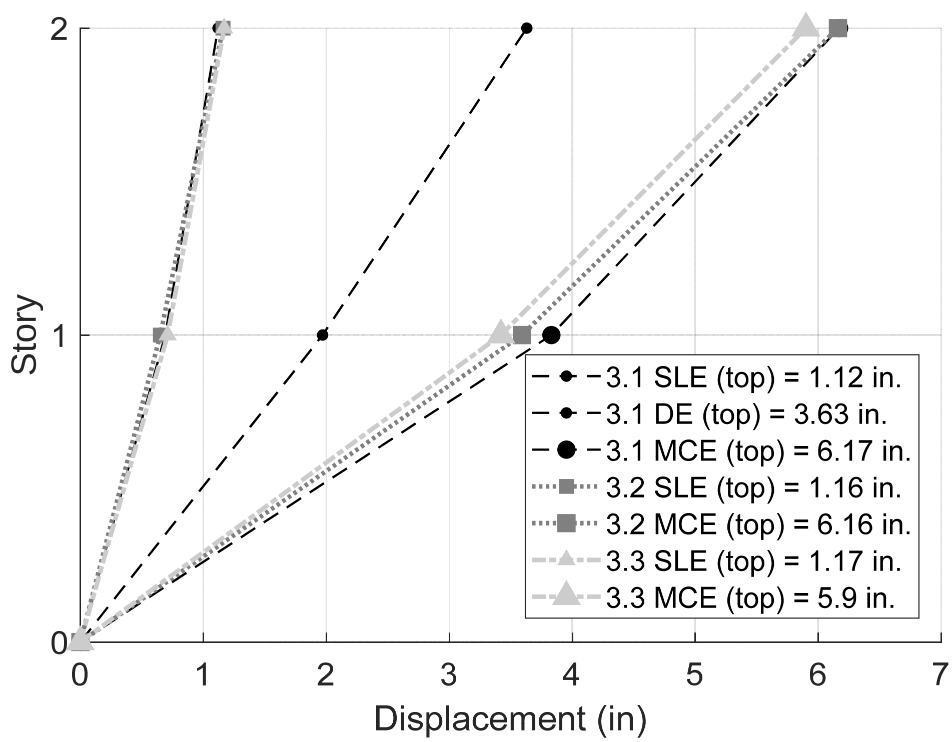


Figure 4.69: Deformation shapes of the structure under different tests

Global hysteresis for all the tests are provided in Figure 4.70-4.76. The base shear was obtained by adding the inertial forces at each floor based on Newton's second law. The average acceleration at each floor was multiplied by the mass corresponding to that floor. Table 29 provides maximum base shear normalized by the weight of the structure for each test.

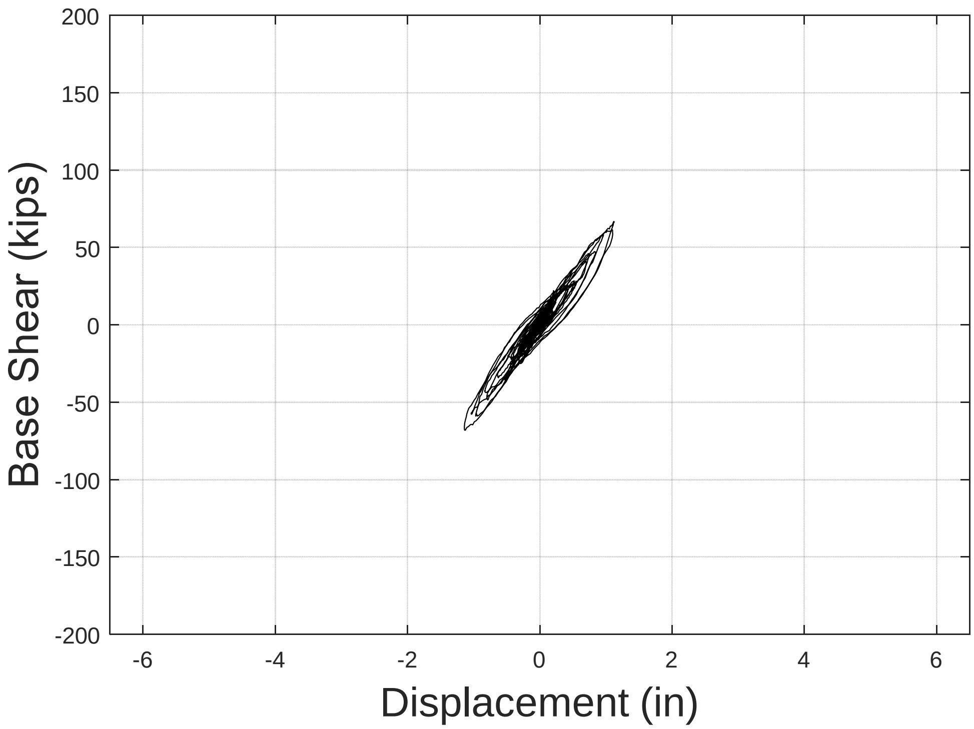


Figure 4.70: Global hysteresis, SLE 3.1

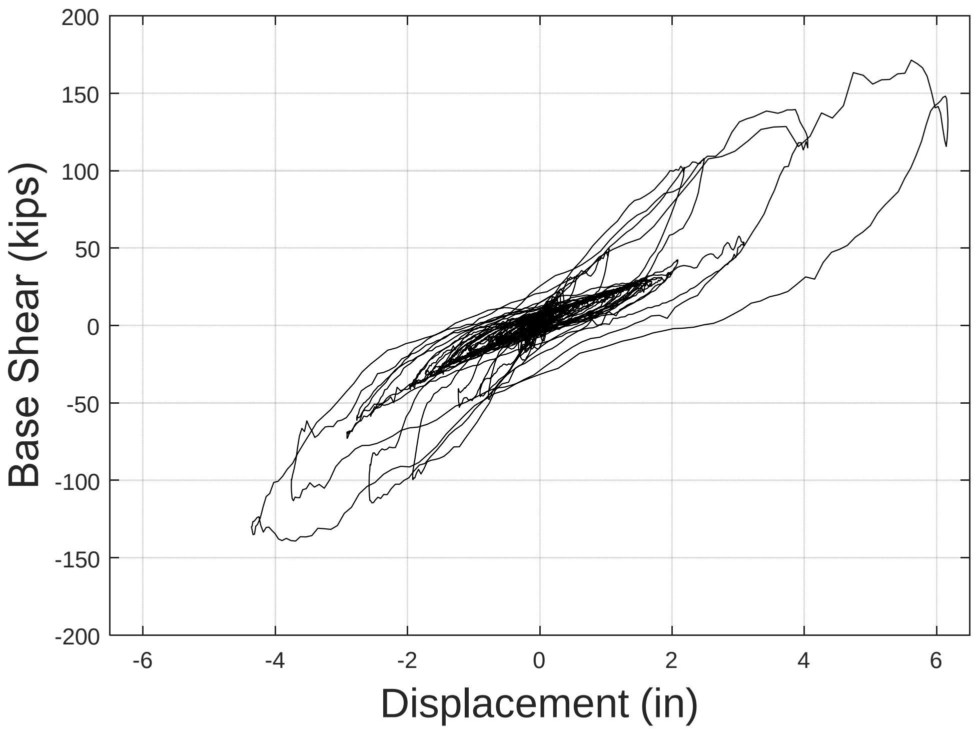


Figure 4.71: Global hysteresis, MCE 3.1

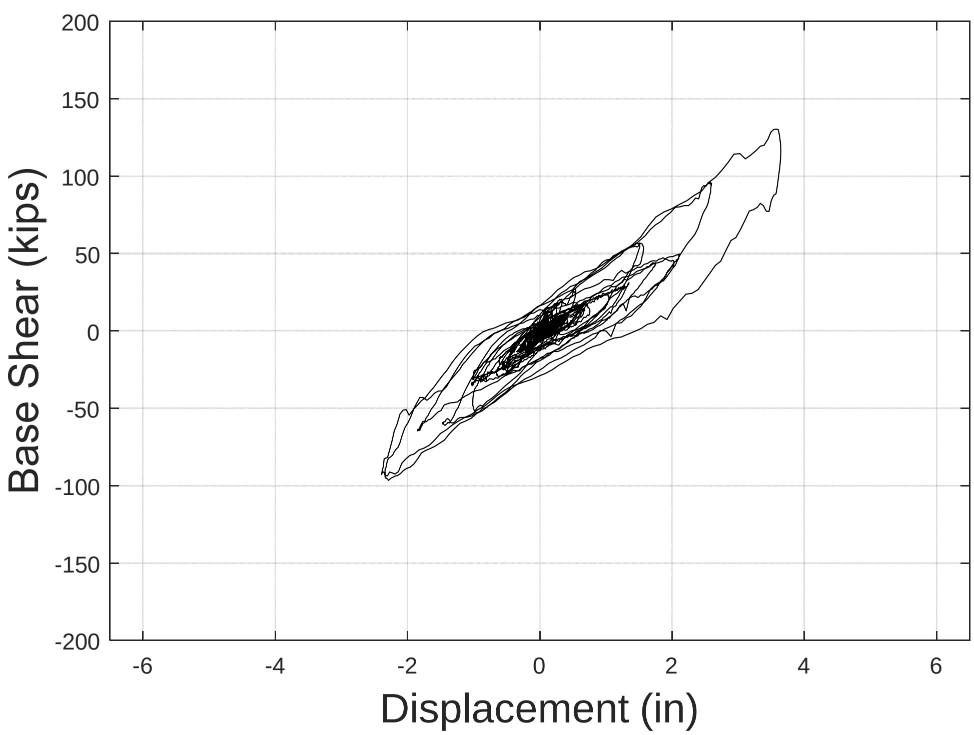


Figure 4.72: Global hysteresis, DE 3.1

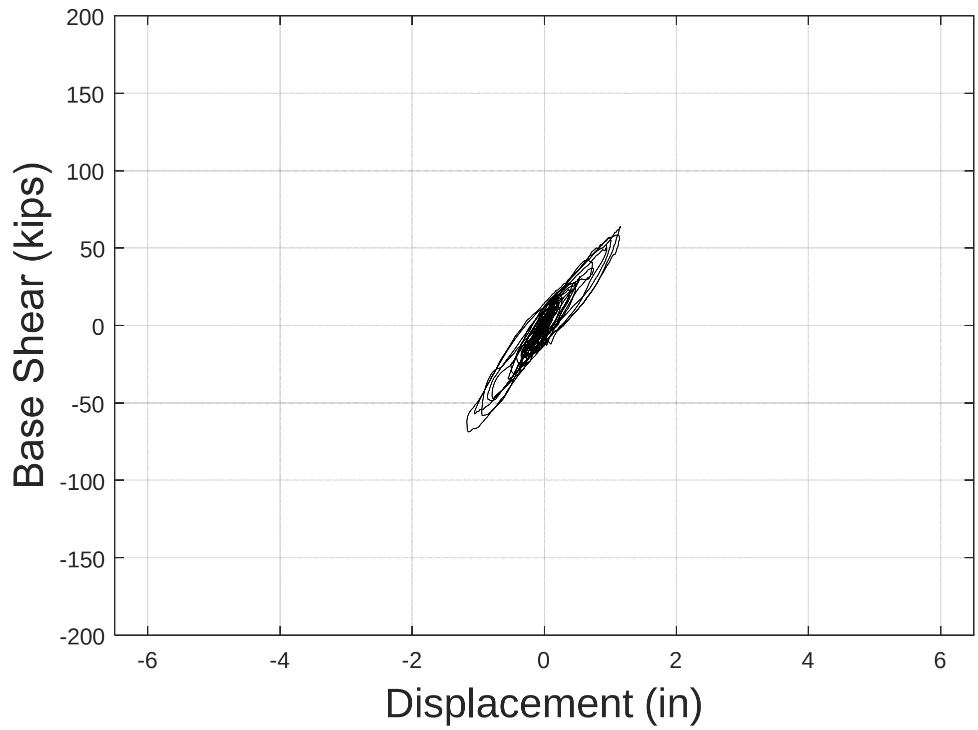


Figure 4.73: Global hysteresis, SLE 3.2

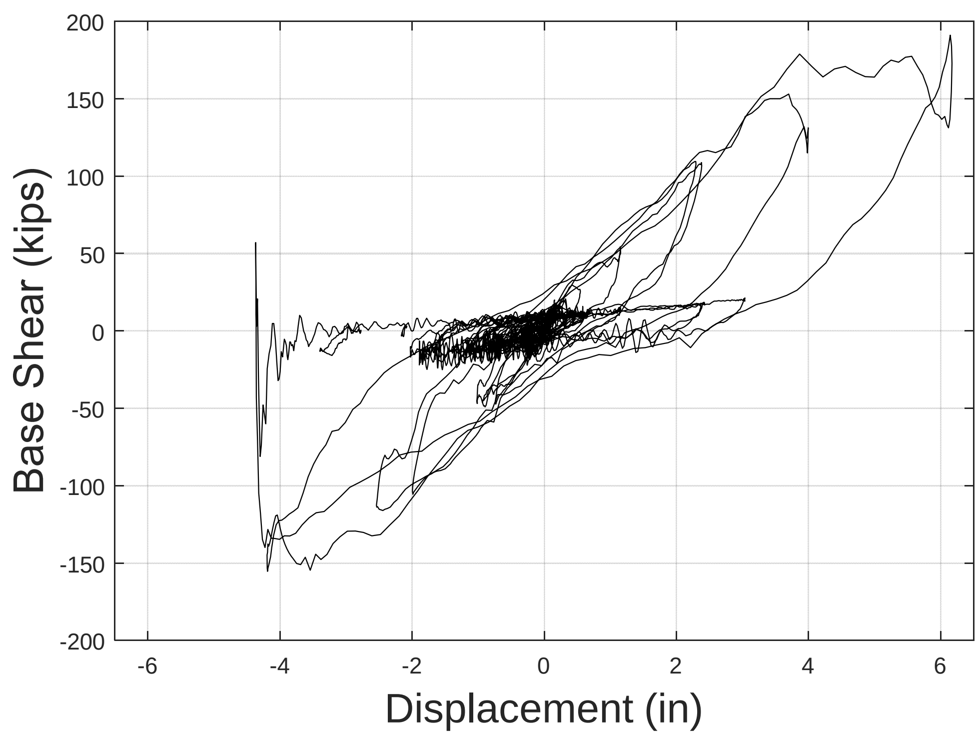


Figure 4.74: Global hysteresis, MCE 3.2

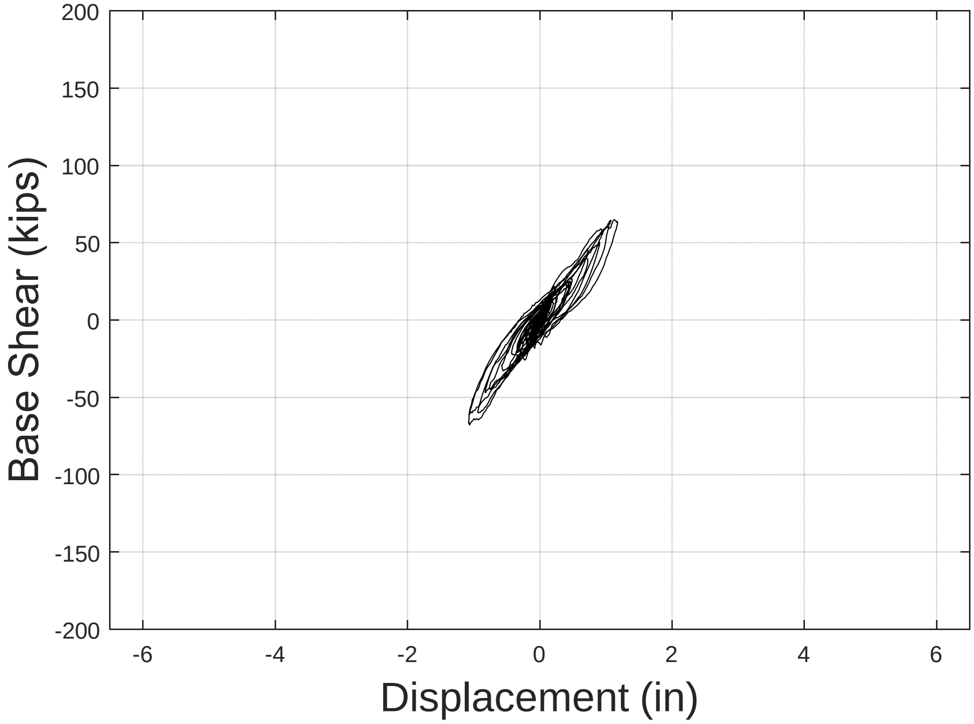


Figure 4.75: Global hysteresis, SLE 3.3

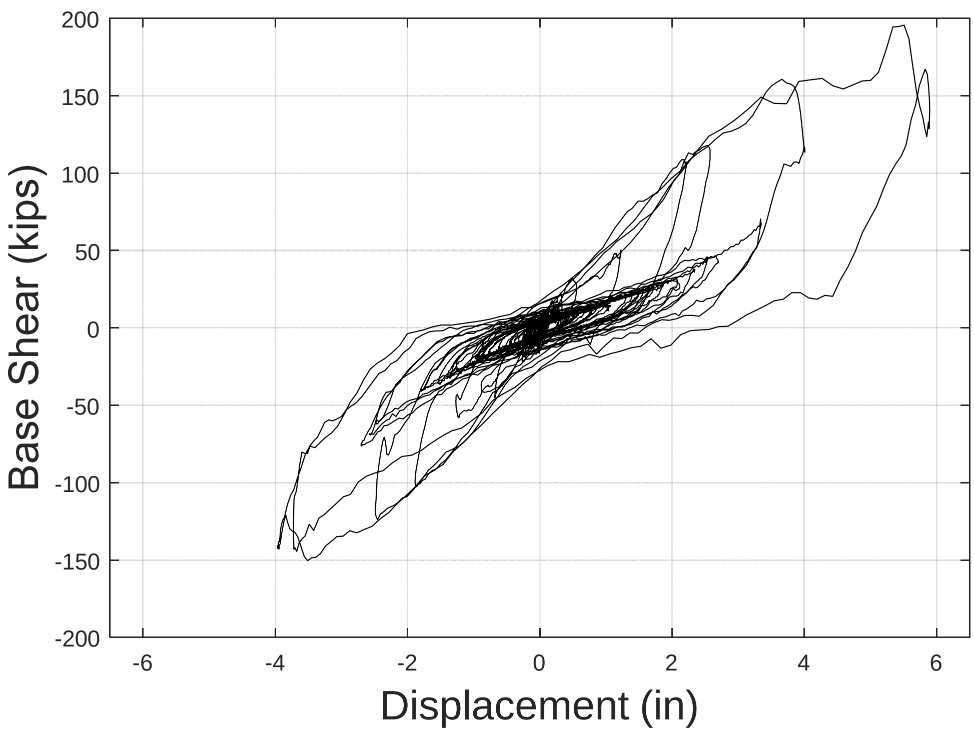


Figure 4.76: Global hysteresis, MCE 3.3

Table 4.18. Summary of force and displacement data

Test Number	Test Configuration	Ground Motion	Story	Inter-story drift(in.)	Drift (%)*	Max Force (kip)**	Base Shear, V_b (kip)	V_b/W***
02	1	Loma Prieta SLE	1	0.69	0.47	27.55	68.11	0.40
			2	0.43	0.36	45.99		
05		Loma Prieta MCE	1	3.83	2.61	70.89	171.4	1.00
			2	2.34	1.95	129.82		
08		Loma Prieta DE	1	1.97	1.34	46.09	130.3	0.76
			2	1.66	1.38	101.84		
11	2	Loma Prieta SLE	1	0.65	0.44	27.24	68.71	0.40
			2	0.51	0.42	48.54		
13		Loma Prieta MCE	1	3.59	2.44	97.45	191	1.11
			2	2.57	2.14	139.06		
16	3	Loma Prieta SLE	1	0.71	0.48	27.96	67.92	0.39
			2	0.46	0.39	42.82		
18		Loma Prieta MCE	1	3.42	2.33	88.31	195.6	1.14
			2	2.48	2.07	144.99		

*Inter-story drift % was calculated based on 1st story height of 12 ft and 3 in. and 2nd story height of 10 ft.

**It is important to note that these maximum forces for each floor do not occur at the same time; therefore, the sums of these two are not equal to maximum base shear shown in the table.

***This ratio was calculated for a first floor weight of 77 kip and second floor weight of 95 kip

4.3.4.2 Diaphragm Behavior

Three string potentiometers were provided to measure movement of the floor and roof diaphragms in the direction of the shaking. These instruments were placed at the north end, south end and at the location of the north wall. Roof and floor displacement for all the tests are provided in Table 4.19. Some torsional behavior observed in all the tests and to varying degrees. The maximum difference between the measurement at the north and south end were 1.41 in. and the minimum was 0.02 in. which were recorded for Tests 05 and 13, respectively. Figures 4.77-4.79 provide a comparison of the north and south measurement for MCE Tests 05, 13, and 18 at the roof level.

Recall that Test Configuration 1 and 3 were both performed with 4:1 aspect ratio panels. However, perpendicular walls were added in the latter case to investigate their effect on the global response. Looking at the data in Table 4.19 and comparing the results from Test 05 and Test 18, one can see that there is less variability between the north end and south end measurements when perpendicular walls were added and as expected these walls reduced torsional behavior of the system.

Table 4.19. Diaphragm displacement measurements

Test Number	Test Configuration	Ground Motion	Story	Displacement (in.) North End	At the North Wall	South End
02	1	Loma Prieta SLE	1	0.63	0.67	0.95
			2	1.05	1.08	1.66
05		Loma Prieta MCE	1	3.73	3.74	4.68
			2	5.94	6.09	7.35
08		Loma Prieta DE	1	1.79	1.79	2.64
			2	3.42	3.58	4.55
11	2	Loma Prieta SLE	1	0.78	-	0.5
			2	1.37	-	1.05
13		Loma Prieta MCE	1	3.61	-	3.59
			2	6.3	-	6.04
16	3	Loma Prieta SLE	1	0.85	-	0.53
			2	1.44	-	0.98
18		Loma Prieta MCE	1	3.54	-	3.35
			2	6.05	-	5.75

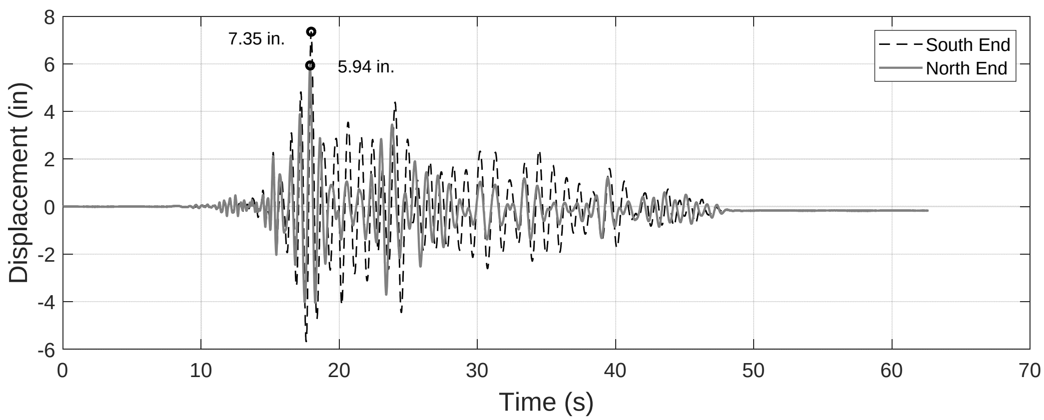


Figure 4.77: Roof diaphragm displacement, Test 05

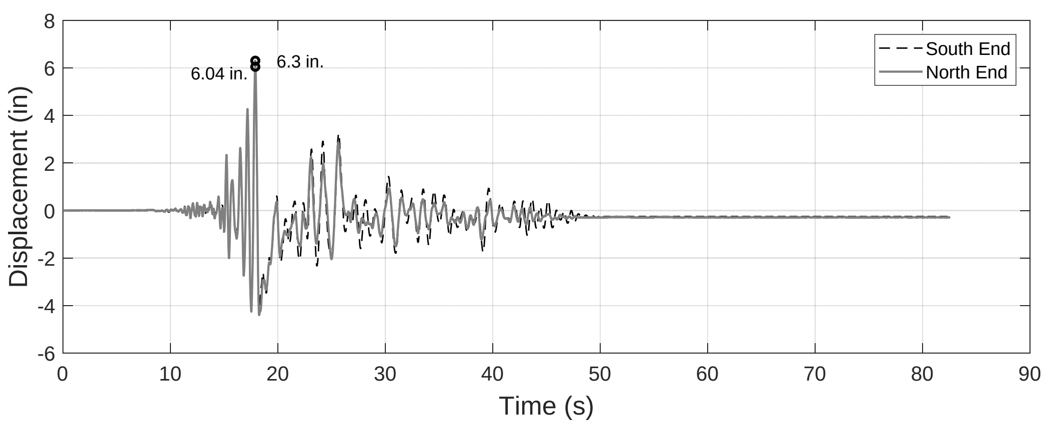


Figure 4.78: Roof diaphragm displacement, Test 13

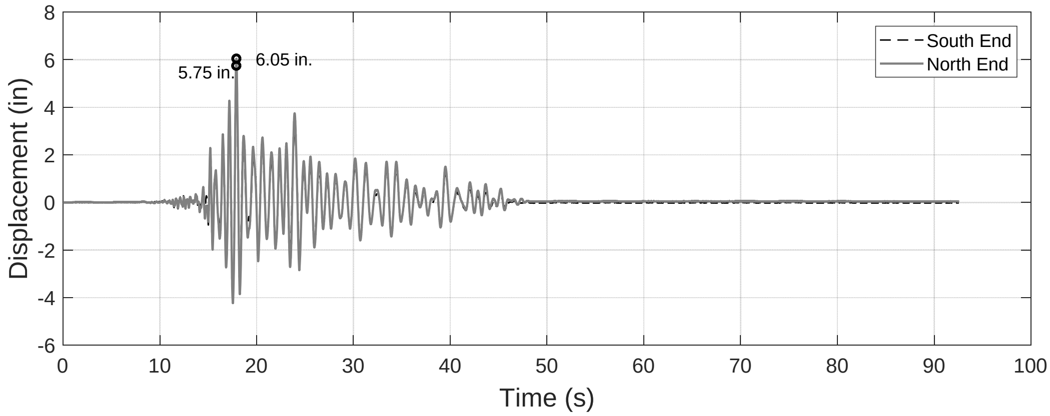


Figure 4.79: Roof diaphragm displacement, Test 18

4.3.4.3 Damage Inspection and Failure Mechanisms

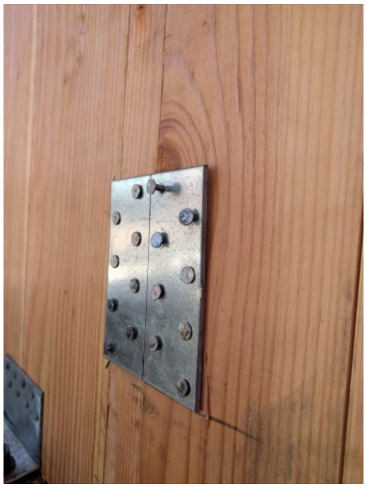
A methodical damage inspection was conducted after each test to determine the effect of ground motion on various aspects of the system. Given the immense nature of the project, a detailed discussion of the damage inspection is not warranted here and the discussion is only focused on the damage and failure mechanism pertinent to the CLT shear wall system utilized in Phase 3 of the project.

All the testing configurations performed well under SLE ground motions and no damage was observed to any of the walls in either floors. The damage under MCE ground motions for all the configurations was mainly concentrated in the base connectors attaching the wall to the supporting element (steel beam) used to transfer the shear load.

For the test configurations 1 and 3 the damage was due to a combination of sliding and mainly rocking. Fastener yielding and withdrawal were observed with no signs of fatigue and Inter-panel connectors (connectors along the vertical joint) also showed nail withdrawal. These are shown in Figures 4.80 and 4.81. In test configuration 3, brackets in the perpendicular walls were undamaged after the tests and there was only slight withdrawal observed which is shown in Figure 4.82.

The failure mechanism in test configuration 2, while initially a combination of rocking and sliding, was mainly due to the sliding. This was expected due to the aspect ratio of the panel. Some yielding and withdrawal of the nails were observed followed by shear failure of the nails due to sliding of the panels and is presented in Figure 4.83.

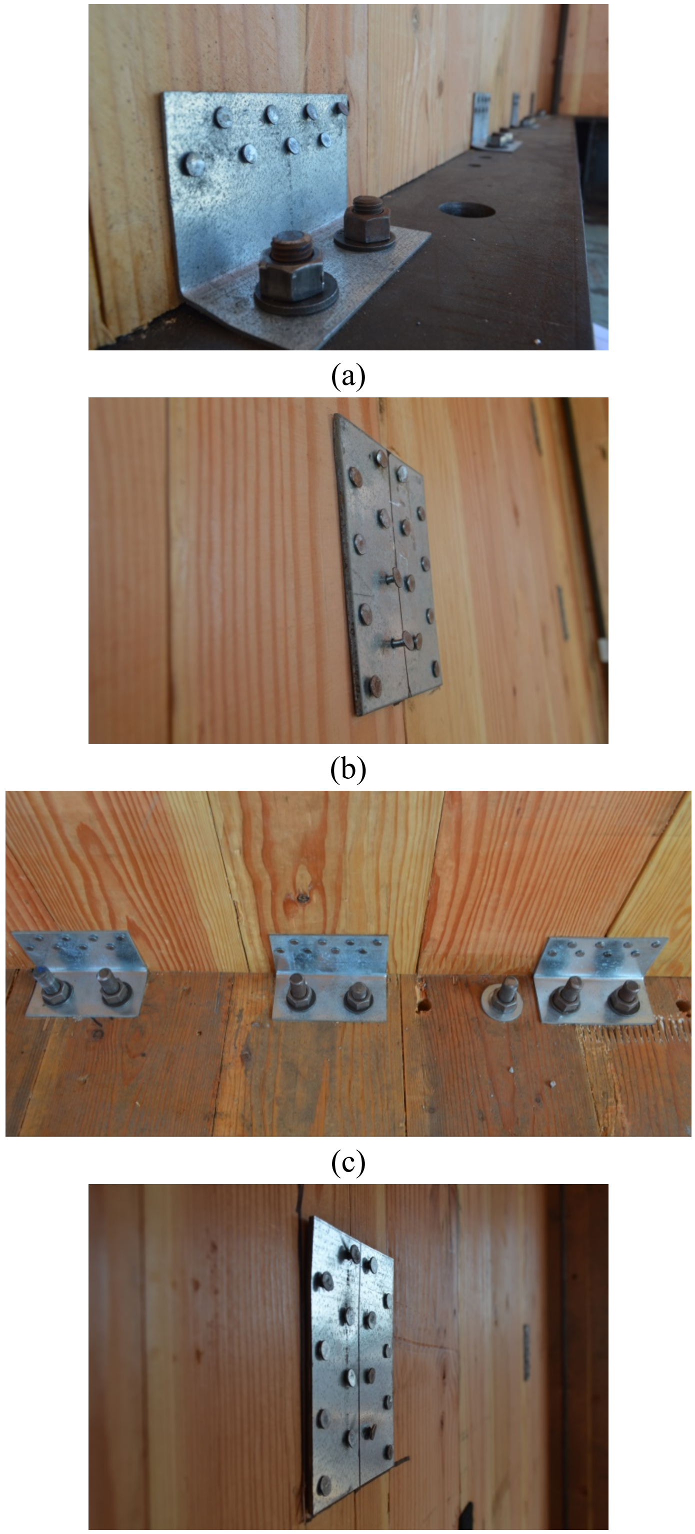
(a)



(b)

(c)

Figure 4.80: Test Configuration 1, Loma Prieta MCE (a) Base connector (b) First floor inter-panel connector (c) Second floor inter-panel connector



(a)

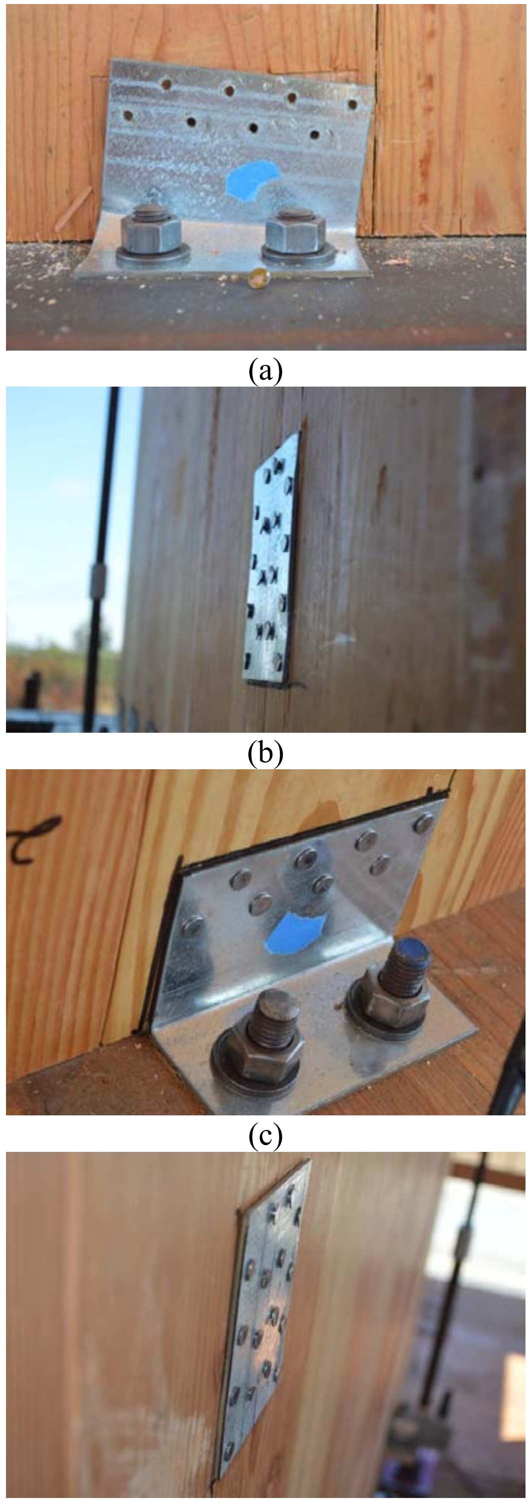
(b)

(c)

(d)

Figure 4.81: Test Configuration 3, Loma Prieta MCE (a) Base connector (b) First floor inter-panel connector (c) Second floor base connector (d) Second floor inter-panel connector

Figure 4.82: Test Configuration 3, Loma Prieta MCE, Perpendicular walls



(a)

(b)

(c)

(d)

Figure 4.83: Test Configuration 2, Loma Prieta MCE (a) Base connector (b) First floor inter-panel connector (c) Second floor base connector (d) Second floor inter-panel connector

4.4 Summary

Testing is one of the major steps identified in the FEMA P695 methodology and this report presents results of the tests conducted at CSU. Tests were performed on the angle brackets and inter-panel connectors. Angle brackets used for connecting the wall to the supporting element were tested under shear and uplift and inter-panel connectors were tested under shear only. These tests were conducted for two different grades of CLT, E1 and V2. The tests were performed under displacement control using CUREE protocol with the reference displacement obtained from monotonic tests. Angle brackets designed per NDS requirement for nailed connections in the vertical leg and bolts in the horizontal leg were used in this project. The A1 connector with a thickness of 3/8 in. did not perform well due to a lack of nail withdrawal and failure was then dominated by nail shear caused by the bracket leg. Connectors A3 and B3 made from 12 gauge steel (0.108 in.) performed as intended and nonlinear behavior was due to the fasteners and some metal connector deformation.

Comparing the results of Tests 05 and 06 showed that cases of with and without the top CLT boundary condition were nominally identical and therefore the remaining tests were conducted without imposing the boundary condition. Tests for gravity loads consisted of the three cases, namely no gravity, 0.68 kip/ft, and 1.28 kip/ft. The results indicated that stiffness and strength both increase as the vertical load increase; however, the change between 0.68 kip/ft and 1.28 kip/ft was less significant. Tests on different grades of CLT, namely E1 and V2, indicated that CLT grade had a significant influence on both stiffness and strength and it is an important parameter. This is very likely the result of specific gravity (i.e. increased density of E1 grade relative to V2 grade) on connection response as well as wood compression deformation and is being investigated.

Tests on two panels with nominally identical designs, except for the panel thickness, one 3.9 in. and the other 9.41 in. thick, showed that thickness has only a slight effect on wall stiffness and strength. This variation was not deemed significant. Other comparisons of the panel behavior based on thickness showed a similar trend. Results of a 4:1 aspect ratio panel compared to 2:1 aspect ratio panel showed the higher aspect ratio panel has significantly less stiffness but has more deformation capacity that the low aspect ratio panel. This increase in deformation capacity can be attributed to the rocking behavior of the panel as opposed to rocking and sliding mechanism of other tested panels. On the other hand, comparing the results of 2:1 with 1:1 aspect ratio panel the differences in stiffness and deformation capacity between the two tests were not as pronounced. Testing has also shown that walls comprised of higher aspect ratio panels that are connected through vertical joints exhibited less stiffness and considerably larger deformation capacity.

Shake table tests were performed on a two-story platform type structure. The structure was assumed to be located in San Francisco and the design forces were calculated based on the Equivalent Lateral Force procedure (ELF) in Section 12.8 of ASCE 7-16 for an R=4 and Φ=0.55. These tests were performed in three phases with each phase corresponding to a different panel configuration. Phase 3.1 and 3.2 were multi-panel configurations with 4:1 (h/b) and 2:1 aspect ratio panel, respectively. Phase 3.3 is similar to Phase 3.1 with the transverse walls added to investigate their effect on shear walls.

All the tests were conducted using the Loma Prieta ground motion record scaled to one of three different hazard levels, namely a service level earthquake (SLE), a design earthquake (DE), and the maximum considered earthquake (MCE), for the site in San Francisco, CA. White noise tests showed the initial period to be larger than the code calculated period which signifies the importance of a period formula for CLT shear wall systems. The structure exhibited linear deformation profile for all the tests. The displacement measurement at the north and south ends of the floor and roof levels documented the torsional behavior that was observed in all the tests with varying degrees. Comparing Phase 3.3 and 3.1 the transverse walls were noted to reduce the story drift as well as the any torsional effects in the system.

There was no damage observed during the SLE tests and the damage in the MCE tests was primarily in the base connectors. Phase 3.1 and 3.3 showed a combination of sliding and rocking where fastener yielding and its subsequent withdrawal was also observed. Phase 3.2, on the other hand, initially showed sliding and rocking which eventually led to the failure mechanism mainly due to sliding. Some fastener yielding and withdrawal was observed which was followed by shear failure of the nails due to sliding behavior of the panels.

CHAPTER 5: DEVELOPMENT OF THE DESIGN METHODOLOGY

This chapter discusses development of the design methodology in terms of connector design, CLT shear wall design and system design. Discussion presented in this chapter leads to the design requirements for CLT shear wall system presented in the next chapter.

5.1 Connector Design

Connector design was informed based on the literature review presented in Chapter 2 of this document. Based on the studies previously summarized, it is generally concluded that CLT panels exhibit linear elastic behavior and energy dissipation occurs at the connector.

Nails are the most common fastener used in wood (FPL, 2010). Fasteners' behavior in wood has been extensively studied and there are a number of parameters that affect their behavior which are mainly categorized as materials and dimensions of the joint components, joint configuration, and loading conditions (Ni, 1997). In the case of CLT, additional factors to consider include presence of adhesives, cross layers, and gaps between the laminations. There has been an increase in recent studies concerning connectors in CLT with the study by Uibel and Blasβ (2006, 2007) at the University of Karlsruhe, Germany, being the most comprehensive performed to date. An overview of the design approach in various standards worldwide to determine connection resistance in CLT is provided by Mohammad et al. (2017). Based on NDS 11.1.1, provisions used to calculated connector resistance for sawn lumber are also applicable to CLT. However, additional criteria are added primarily to account for cross layers of CLT.

Based on Gavric et al. (2014) and considering the proposed angle bracket layout, various possible failure modes include: shear failure of the nails, group tear out of the fasteners, net section failure of the steel part of the angle bracket in shear and tension for the horizontal and vertical leg, respectively, block shear of the steel part of the angle bracket, pull-through shear failure of the connector horizontal leg, and tension and shear of the bolts attaching the bracket to the supporting element. When the angle brackets and attaching anchor bolts are sized properly and spacing and other requirements are met to avoid the type of failures discussed before, development of single fastener yield modes can occur and the connector capacity can be taken as the sum of all individual fasteners.

The fastener resistance is calculated based on NDS yield equations. The yield equations consider various yield modes of wood bearing and nail bending. Mode I_s and I_m are based on the bearing yield limit state in side member and main member, respectively. Mode II is for rotation of the fastener without bending and with localized crushing of the wood. Mode III and IV are a combination of formation of either one or two plastic hinges and wood crushing. These modes are shown in Figure 5.1 and the corresponding equations based on NDS provided in Table 5.1.

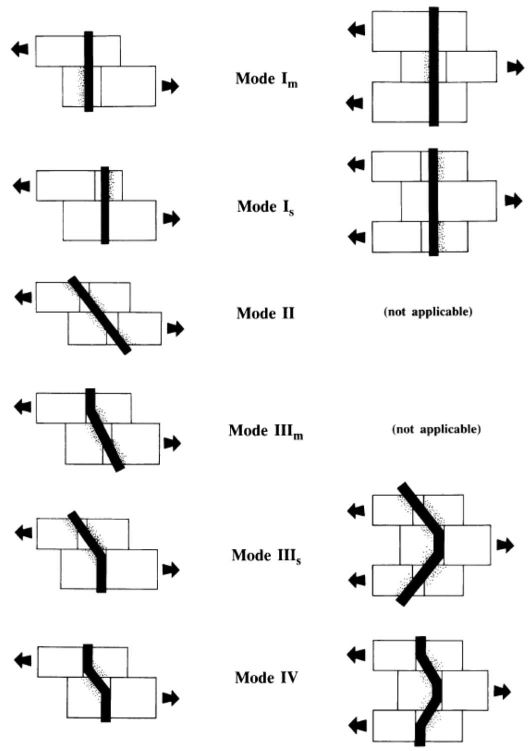


Figure 5.1: Connection Yield Modes (ANSI/AWC, 2015)

Table 5.1. Yield Limit Equations

Yield Mode	Yield Limit Equations, Single Shear
I_m	$Z = \frac{D\ l_m\ F_{em}}{R_d}$
I_s	$Z = \frac{D\ l_s\ F_{es}}{R_d}$
II	$Z = \frac{k_1\ D\ l_s\ F_{es}}{R_d}$
III_m	$Z = \frac{k_2\ D\ l_m\ F_{em}}{(1+2R_e)R_d}$
III_s	$Z = \frac{k_3\ D\ l_s\ F_{em}}{(2+R_e)R_d}$
IV	$Z = \frac{D^2}{R_d}\sqrt{\frac{2\ F_{em}\ F_{yb}}{3\ (1+R_e)}}$

where

$$k_1 = \frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)}$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{F_{yb}\,(1 + 2R_e)D^2}{3F_{em}\,l_m^2}}$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}\,(2 + R_e)D^2}{3F_{em}\,l_s^2}}$$

D= diameter, in.
F_{yb}= dowel bending yielding strength, psi.
R_d= reduction term, $R_d = K_D$ for D<0.25 in.

$K_D = 2.2$ for D≤0.17 in.
$K_D = 10D+0.5$ for 0.17 in. < D <0.25 in.

$R_e= F_{em}/F_{es}$
$R_t= l_m/l_s$
l_m= main member dowel bearing length, in.
l_s= side member dowel bearing length, in.
F_{em}=main member dowel bearing strength, psi.
F_{es}= side member dowel bearing strength, psi.

Based on Section 11.3 of NDS, reference design value, Z, shall be adjusted using the following factors:

C_M= Wet Service Factor

C_t=Temperature Factor

C_g=Group Factor

C_Δ=Geometry Factor

C_{eg}=End Grain Factor

C_{di}=Diaphragm Factor

C_{tn}=Toe-Nail Factor

The adjusted design value, Z', is calculated using the following equation:

$$\boldsymbol{Z' = Z * C_D * C_M * C_t * C_g * C_\Delta * C_{eg} * C_{di} * C_{tn}} \quad \textbf{(5.1)}$$

Shear design calculations for A3 type connectors are provided below:

CLT Connection Design

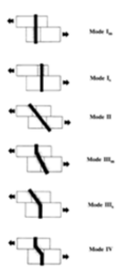
INPUT DATA & DESIGN SUMMARY

NAIL TYPE (0=Lag screw, 1=Wood screw, 2=Bolts, 3= Nails)		3	Nail
Number of fasteners in the connection	n_F=	16	
Number of shear planes per nail	n_S=	1	
Load Duration Factor	C_D	1.00	
Wet Sevice Factor	C_M	1.00	
Temperature Factor (Ft, E)	C_t	1.00	
Group Action Factor	C_g	1.00	
Geometry Factor	C_Δ	1.00	
End Grain Factor	C_{eg}	1.00	
Diaphram Factor	C_{di}	1.00	
Toe-Nail Factor	C_{tn}	1.00	

		NDS	
THE ALLOWABLE LATERAL DESIGN VALUE FOR THE 8 NAILS IN THE CONNECTION IS	=	**1570**	**lb.**
THE LRFD NOMINAL VALUE FOR THE 8 NAILS IN THE CONNECTION IS	=	**5216.7**	**lb.**

ANALYSIS

Nail diameter	D=	0.135	in.	
Specific Gravity	G=	0.42		
Embedding strength of the main member	f_{em}=	3364	psi	
Embedding strength of side member	f_{es}=	61850	psi	for ASTM 653
Main member dowel bearing length	l_m	3.3980	in.	
Side member dowel bearing leangth	l_s	0.1050	in.	
Nail yield strength	f_{yb}=	100000	psi	
Density of cross-laminated timber	D=	420	kg/m^3	
Characteristic embedment strength	$f_{h,k}$	34.4	N/mm^2	
		4983.8	psi	



Unit lateral resistance		Solid Wood	Reduction term (Rd)
I_m	$Z=\frac{Dl_mF_{em}}{R_d}$	701.5 lb.	2.20
I_s	$Z=\frac{Dl_sF_{es}}{R_d}$	398.5 lb.	2.20
II	$Z=\frac{k_1Dl_sF_{es}}{R_d}$	286.4 lb.	2.20
III_m	$Z=\frac{k_2Dl_mF_{em}}{(1+2R_e)R_d}$	293.6 lb.	2.20
III_s	$Z=\frac{k_3Dl_sF_{em}}{(2+R_e)R_d}$	98.1 lb.	2.20
IV	$Z=\frac{D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$	120.8 lb.	2.20

K_D= 2.2

K_θ= 1

Max angle of load to grain (0° ≤ θ ≤ 90°)

θ= 0 deg.

$R_e=F_{em}/F_{es}$ R_e= 0.054

$R_t=l_m/l_s$ R_t= 32.4

$$k_1=\frac{\sqrt{R_e+2R_e^2(1+R_t+R_t^2)+R_t^2R_e^3}-R_e(1+R_t)}{1+R_e} = 0.719$$

$$k_2=-1+\sqrt{2(1+R_e)+\frac{2F_{yb}(1+2R_e)D^2}{3F_{em}l_m^2}} = 0.464$$

$$k_3=-1+\sqrt{\frac{2(1+R_e)}{R_e}+\frac{2F_{yb}(2+R_e)D^2}{3F_{em}l_s^2}} = 9.299$$

Typical Nail Dimensions

Type	Fyb
Lag Scew	45000
Wood Screw	45000
Bolt	45000
Nail	100000

Technical References:

Figure 5.2. A3 type connector shear design calculations

Similarly, based on Section 12.2 of NDS, reference withdrawal design value for the connector nailing was calculated using the following equations:

$$\boldsymbol{W = 1380\, G^{5/2} D} \tag{5.2}$$

$$\boldsymbol{W' = W * C_D * C_M * C_t * C_{eg} * C_{tn}} \tag{5.3}$$

where D is nail diameter, W is reference design value per unit length of penetration ,W' is the adjusted design value and other factors were defined earlier.

Calculations for A3 type connector nail withdrawal values are shown below:

CLT Connection Design

INPUT DATA & DESIGN SUMMARY

Number of fasteners in the connection	n_F=	8	
Load Duration Factor	C_D=	1.00	
Wet Sevice Factor	C_M=	1.00	
Temperature Factor (Ft, E)	C_t=	1.00	
End Grain Factor	C_{eg}=	1.00	
Toe-Nail Factor	C_{tn}=	1.00	
Nail length	ℓ=	3.50	in.
Side member thickness	t_s=	0.105	in.

		NDS	
THE ALLOWABLE WITHDRAWAL DESIGN VALUE FOR THE 8 NAILS IN THE CONNECTION IS	=	578	lb.
5 TIMES THE ALLOWABLE WITHDRAWAL DESIGN VALUE FOR THE 8 NAILS IN THE CONNECTIONS IS*	=	2892	lb.
THE LRFD NOMINAL VALUE FOR THE 8 NAILS IN THE CONNECTION IS	=	1922.2	lb.

ANALYSIS

Nail diameter	D=	0.135	in.
Specific Gravity	G=	0.42	

ASD fastener nail withdrawal per length of penetration

$W = 1380 G^{5/2} D$ = 21.30 lb/in.

ASD fastener nail withdrawal

$W * l =$ = 72.31 lb

Technical References: NDS 12.2.3

* In this case the LRFD nominal is not the best estimate of the peak strength of the connection in withdrawal.
Based on the original withdrawal testing, described in NDS Commentary, withdrawal test values are 5x withdrawal design value.

Figure 5.3. A3 type connector withdrawal design calculations

It is important to note that for the purpose of design, a designer would not be checking withdrawal of the fasteners for the shear connector. In this particular case, A3 would be treated as a shear-only connection and the value of interest will be reference lateral design value, Z. However, considering the connector configuration and the test results, the strength limit state and overall connection deformation capacity is in part governed by nail withdrawal since combined lateral load and nail withdrawal was the observed mode of failure. Therefore, withdrawal design value calculations are provided to help understand the overall performance.

As mentioned earlier when other undesirable modes of failure are avoided and development of yield modes is ensured, the connector design capacity is taken as the sum of all individual fasteners. Therefore, LRFD nominal capacity for the B3 connector was taken as double the capacity of an A3 connector.

Comparison of the connector test values presented in Section 4.1 with the design values determined in accordance with NDS for the nails connecting the angle bracket to the CLT are provided in Table 5.2.

Table 5.2. Comparison of the test with the design values

Test type	Connector Type	CLT Grade	Avg. of the Max.Test Value (kip)*	Nominal Design Value (kip)	Avg. Test Value/Nominal Design Value
Shear	A3	V2	3.41	2.605	1.31
		E1	3.82		1.47
	B3	V2	5.51	5.210	1.06
		E1	6.42		1.23
	E3	V2	3.14	2.605	1.21
		E1	3.71		1.42
Uplift	A3	V2	3.80	2.605	1.46
		E1	3.82		1.47
	B3	V2	7.63	5.210	1.46
		E1	8.83		1.69

*These values are based on Table 13, 14, and 15

Currently there aren't any provisions in the American Institute of Steel Construction (AISC) manual (2017) addressing shear design of angle brackets. Behavior and design of cold formed steel angle brackets was studied by Yu et al. (2016). Based on the connector configuration, the study adopted AISC provisions (2011) for double coped beam as a reference design method for cold form steel brackets. However, design based on these provisions did not show good agreement with the test results. Two sets of equations were proposed by the authors for nominal shear strength for cases both with and without consideration of deformation of the angle bracket. The proposed equations were considered valid for the following properties and boundary conditions:

- Angle bracket thickness 0.033-0.097 in. (0.84-2.46 mm)
- Angle bracket design yield strength 33-50 ksi (227-345 MPa.)
- L/B ratio of 0.18-1.40, where L is distance between the first line of the screws to the bend line and B is connector width
- The fastener pattern shall allow full engagement of the vertical leg under shear load

With the exception of slightly larger connector thickness of 0.108 in., other criteria are met for A3 and B3 type connector calculations with their respective design calculations based on the equations provided in Yu et al. (2016) are provided herein. The calculated values are nominal capacity where ϕ=0.57 (LRFD) and Ω=2.78 (ASD) and ϕ=0.53(LRFD) and Ω=3.02 (ASD) are proposed for cases of with and without consideration of deformation, respectively. It is important to note that the referenced research only focused on failure in the cantilever leg of the angle bracket and other failure modes including fastener failure were not included. Conversely, as mentioned earlier, in this study the primary failure mode is the fastener yielding which is desired due to the ductility associated with it. While these calculations were not used in the final design, they did however serve as a check to ensure the metal part of the angle bracket had adequate capacity to avoid an undesired connector failure mechanism.

Calculations for A3 connector:

CLT Connection Design

INPUT DATA & DESIGN SUMMARY

Distance between the center of first line of screws to the bend line	L=	1.75	in.	
Connector width	B=	4.75	in.	
Connector thickness	t=	0.108	in.	
Material yield stress	Fy=	33000	psi	ASTM 653 Grade 33
Modulus of Elasticity	E=	29000000	psi	
Poisson ratio	μ=	0.3		

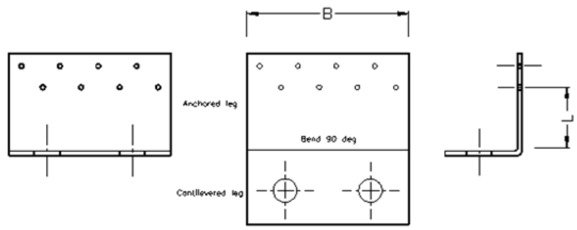
$k = 2.569(\frac{L}{B})^{-2.202}$ = 23.16

$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}(\frac{t}{B})^2$ = 313768.3 psi

$V_y = 0.6F_y Bt$ = 10157.4 lb

$V_{cr} = F_{cr} Bt$ = 160963.1 lb

$\lambda = \sqrt{\frac{V_y}{V_{cr}}}$ = 0.251



Nominal shear strength without consideration of deformation

$V_n = 0.231\lambda^{-0.8}V_y \leq 0.583V_y$ = **5921.8 lb**

Nominal shear strength with consideration of deformation

$V'_n = 9000\alpha\left(\frac{Bt}{L}\right) \leq V_n$ = **2638.3 lb**

α= 1 for SI units

Figure 5.4. A3 type connector calculations based on Yu et al. (2016)

Calculations for B3 connector:

CLT Connection Design

INPUT DATA & DESIGN SUMMARY

Distance between the center of first line of screws to the bend line	L=	1.50	in.	
Connector width	B=	5.56	in.	
Connector thickness	t=	0.108	in.	
Material yield stress	Fy=	33000	psi	ASTM 653 Grade 33
Modulus of Elasticity	E=	29000000	psi	
Poisson ratio	μ=	0.3		

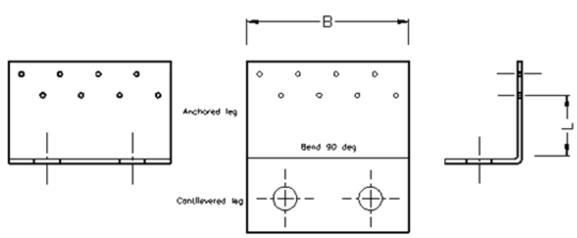
$k = 2.569(\frac{L}{B})^{-2.202}$ = 46.04

$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}(\frac{t}{B})^2$ = 454860.6 psi

$V_y = 0.6F_y Bt$ = 11894.9 lb

$V_{cr} = F_{cr} Bt$ = 273257.5 lb

$\lambda = \sqrt{\frac{V_y}{V_{cr}}}$ = 0.209



Nominal shear strength without consideration of deformation

$V_n = 0.231\lambda^{-0.8}V_y \leq 0.583V_y$ = **6934.7 lb**

Nominal shear strength with consideration of deformation

$V'_n = 9000\alpha\left(\frac{Bt}{L}\right) \leq V_n$ = **3604.5 lb**

α= 1 for SI units

Figure 5.5. B3 type connector calculations based on Yu et al. (2016)

5.2 CLT Shear Wall

CLT special shear walls are in single panel or multi-panel configurations and the detailing incorporates panel aspect ratio limits and prescribed unit shears for prescribed connectors at bottom of panel, top of panel and at vertical joints of multi-panel shear walls. Design unit shears are associated with uniform spacing of these connectors at each of these locations. These typical configurations are shown in Figures 5.6 and 5.7. Multi-panel shear walls are formed by individual panels of the same aspect ratio to promote deflection compatibility within the shear wall.

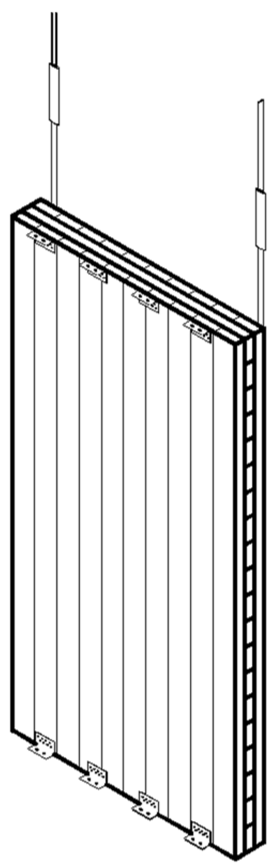


Figure 5.6: Typical single panel configuration

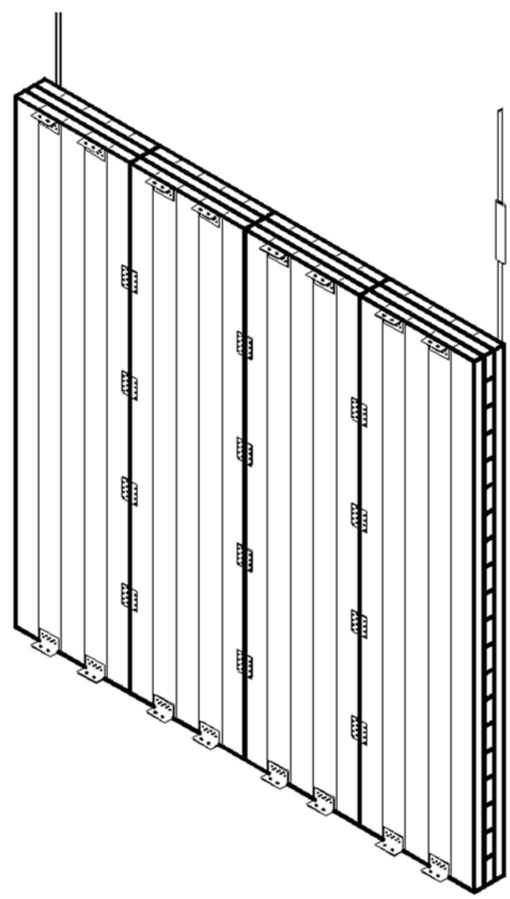


Figure 5.7: Typical multi-panel configuration

5.2.1 Design

Currently, there are no standard models for determining lateral capacity of a CLT shear wall. As mentioned earlier, CLT panels individually demonstrate rigid behavior and deformation and energy dissipation occurs at the connectors. This forms the basis of various proposed models where assumptions vary in terms of connector behavior that include shear only, uplift only and a combination of shear and uplift. An overview of various analytical models is presented in Garvic and Popovski (2015).

The kinematic model used for the purpose of this study is presented in Figure 5.8. The main design assumption is that angle brackets resist shear only and holddowns are provided at the ends of the wall for overturning restraint. It should be noted that this type of assumption was also utilized in the initial stages of the Italian SOFIE project (Ceccotti, 2008). However, subsequent studies (Gavric et al., 2015b; Rinaldin and Fragiacomo, 2016) have shown the axial contribution of the connectors to be significant. Regardless, the approach adopted in the FEMA P695 study and its design approach aligns well with the already established methods for light-frame wood shear walls and a comparison of different analytical models with experimental data (Gavric et al., 2015a) showed this approach to be conservative. In addition, this assumption, while conservative, will provide designers an easier application in design.

Aspect ratio limits are imposed on CLT panels that form either single or multi-panel shear walls. In addition to the maximum and minimum aspect ratio requirements, prescribed connectors and connector spacing requirements, other requirements such as those for the design of the overturning device and compression zone ensure that shear capacity is developed in the multi-panel configurations while promoting a rocking behavior. The design intent of the methodology aligns well with tested performance.

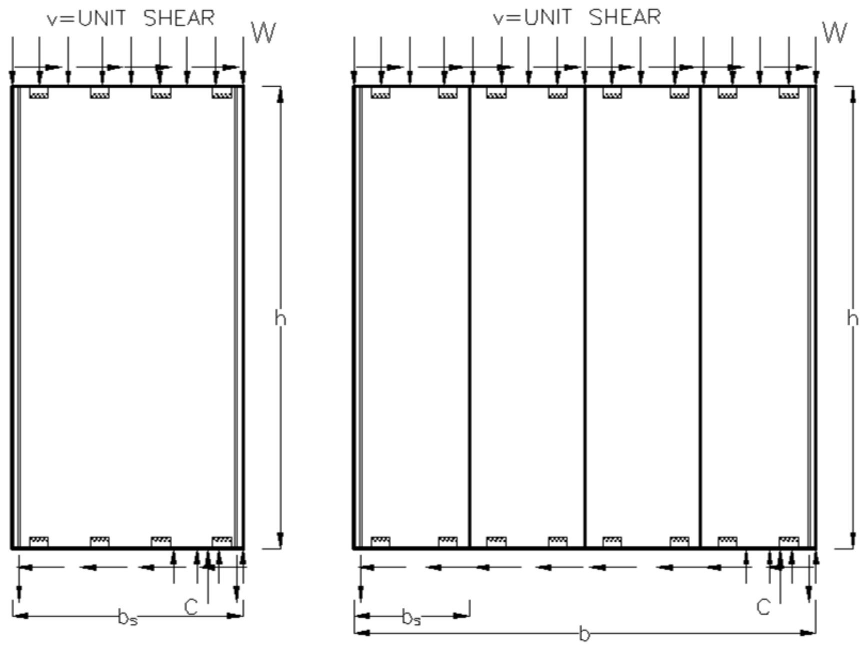


Figure 5.8: Kinematic model for single and multi-panel configuration

5.2.2 In-plane shear at top and bottom of shear wall

Prescribed connectors require nails in the vertical leg and bolts in the horizontal leg. Required connector thickness is 0.105 in (12 Gage ASTM A653 Grade 33). The combination of nails and connector thickness is not subject to modification without verification by testing. Underlying tests utilizing fully-reversed cyclic loading of greater connector thickness, 0.375 in., showed occurrence of nail failure while lesser connector thickness, 0.108 in., was associated with connector failure at the bend between the horizontal and vertical leg. For the Type A3 connector with 8 nails in the vertical leg, observed failure was due to combined nail bending and withdrawal from the wood. For the Type B3 connector with 16 nails in the vertical leg, failure of both nails in combined bending and withdrawal and connector tear was observed. In some cases where connector tear occurred, the location was at the bend between the vertical and horizontal leg and was the ultimate failure following nail bending and partial withdrawal of connector nails from the wood.

Prescribed connectors have been evaluated under fully reversed cyclic testing of shear walls. Additionally, connectors have been tested as components separately under uplift loading and shear loading. Where alternatives to the prescribed nails and connectors are sought, evaluation could first utilize connector testing to screen for strength and stiffness performance and then be followed by shear wall testing to evaluate effects of simultaneous uplift and shear loading experienced by the connection in a shear wall application. Testing employed bolts in the horizontal leg of the connectors. Lag screws are prescribed as an alternative based on calculation to provide equivalent lateral design strength and calculated withdrawal capacity, on an ASD basis, not less than the expected strength of the connector.

5.2.3 Shear transfer at vertical joint in multi-panel shear wall

Required thickness for the vertical joint connector is 0.105 in. and is the same as the angle brackets. It also uses the same nailing as the angle brackets. The nailing and connector thickness is not subject to modification without verification by testing. Connectors for the vertical joints are sought to provide equivalent shear capacity to that of the angle brackets. Type E connector is equivalent to Connector Type A and Type F connector is equivalent to Connector Type B. Type E connector was tested in reverse cyclic testing of shear walls and both Type E and Type F were also tested separately as part of the component testing.

5.2.4 Overturning resistance

5.2.4.1 Load Combinations

The 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).

For LRFD, the applicable load combinations are:

5. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + \rho Q_E$

For ASD, the applicable load combinations are:

5. $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E$
6b. $(1.0 + 0.10S_{DS})D + 0.525\rho Q_E + 0.75L + 0.75S$
7. $(0.6 - 0.14S_{DS})D + 0.7\rho Q_E$

Requirements for tie-down devices to resist shear induced overturning are intended to address two common tie-down systems. For both continuous rod systems and conventional tie-downs, it is required that the strength of the device exceed the expected forces that can be developed.

Predictable wood shear wall performance is achieved when drift contribution from the holddowns are kept minimal (SEAOC, 2008). For both continuous rod systems and conventional tie-downs systems, a specific device elongation limit of 0.18 in. using ASD design is recommended to be met at each level to avoid concentration of deformations in one level. This is based on AC 391 Acceptance Criteria for Continuous Rod Tie-Down Runs and Continuous Rod Tie-Down Systems used to resist wind uplift (2010).

5.2.4.2 *Compression Zone*

The compression zone for wood bearing stress is assumed to be uniformly distributed as depicted in Figures 5.9-5.11. Bearing area must be adequate to contain the compression zone within the outer most panel of the multi-panel wall. Additionally, consistent with rotation behavior of individual CLT wall panels within the shear wall as opposed to the shear wall overturning as rigid monolith, static equilibrium (see Figure 5.11) is based on summing moments about O which represents the tension edge of the compression end panel.

Compression force calculation by static equilibrium of compression end panel (see Figure 5.11):

$$\sum M_o = 0$$

Overturning moment due to factored loads

= *(overturning moment due to unit shear, v , kip/ft)*

+ *(overturning moment due to dead load, w, kip/ft)*

$$= -(\boldsymbol{v} * \boldsymbol{b_s} * \boldsymbol{h}) - \left(\mathbf{1.4} * \boldsymbol{w} * \boldsymbol{b_s} * \frac{\boldsymbol{b_s}}{\mathbf{2}}\right) - \boldsymbol{C_T} * \left(\boldsymbol{b_s} - \frac{\boldsymbol{x_T}}{\mathbf{2}}\right) \qquad \textbf{(5.4)}$$

where

v=unit shear, kip/ft
w= unit gravity including wall panel self-weight, kip/ft
1.4= load factor on D from LRFD combination 5 for assumed value of S_{DS}= 1.0 (see above)
b_s= CLT panel length, ft
h= CLT panel height, ft
C_T= compressive bearing force from the top floor
x_T= length of compression zone at the top of the wall from rocking of the wall above

Resisting moment due to bearing stress under compression end panel

= *(compressive bearing force, C, kips)* (moment arm, ft)*

$$= \boldsymbol{C} * \left(\boldsymbol{b_s} - \frac{\boldsymbol{x}}{\mathbf{2}}\right) \qquad \textbf{(5.5)}$$

where

b_s = CLT panel length, ft
x= length of compression zone, ft

The equation for static equilibrium considering combined moment due to factored loads and resisting moment due to compression bearing resistance is:

$$\boldsymbol{C * \left(b_s - \frac{x}{2}\right) - (v * b_s * h) - (1.4 * w * b_s * \frac{b_s}{2}) - C_T * \left(b_s - \frac{x_T}{2}\right) = 0} \qquad \textbf{(5.6)}$$

Length of compression zone, x, limited by compression bearing resistance (e.g. $= F_{C\perp} * t * x * \frac{12in}{ft}$) is also determined by the following equation:

$$\boldsymbol{x = \frac{C}{F_{C\perp}} * \frac{1ft}{12\ in}} \qquad \textbf{(5.7)}$$

x= length of compression zone, ft
$F_{C\perp}$= LRFD bearing stress perpendicular to grain in CLT floor panel (equal to 0.638 ksi for SPF panels)
t= CLT panel thickness, in.
(1ft/12 in.)= conversion to obtain compression zone length, x, in feet

In the case of compression perpendicular to the grain, in situ conditions are often very different than standard tests specimens (Leijten and Jorissen, 2010) used to evaluate this property. Generally for timber it is dependent on the geometry (Madsen et al., 1982) and particularly in the case of CLT (Serrano and Enquist, 2010) it depends on loading area and orientation. Properties of CLT in compression perpendicular to the grain has been extensively studied by Bogensperger et al. (2011) and Brandner and Schickhofer (2014). For the purpose of this study, compression strength perpendicular to the grain has been taken as 0.638 ksi for SPF panels based on NDS.

Length of compression zone, x, to precisely satisfy static equilibrium is determined by substitution of Eq. 5.6 into Eq. 5.5 and solving for x.

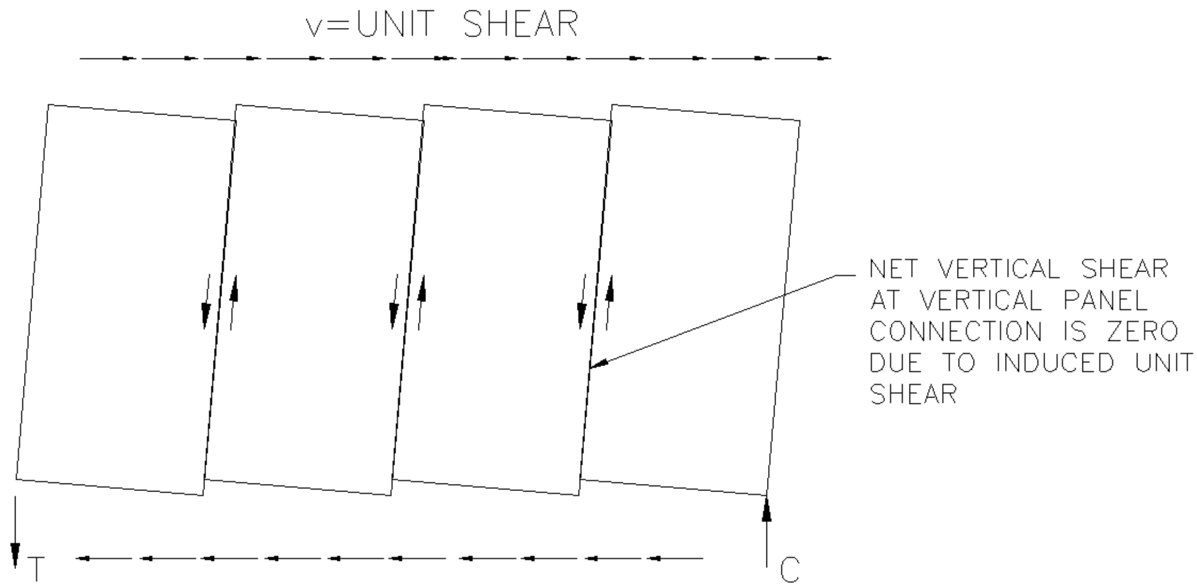


Figure 5.9: Rotation of individual panels in a CLT shear wall

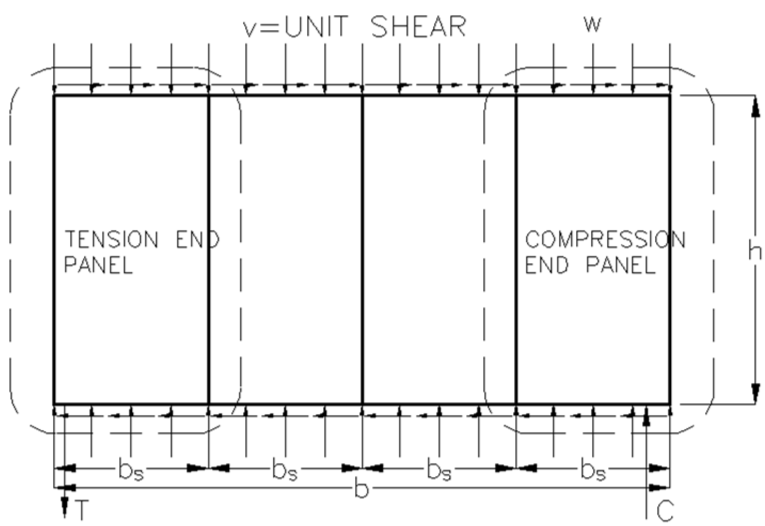


Figure 5.10: Combined shear and gravity loading and reactions for CLT shear wall comprised of multiple CLT panels- compression end and tension end panel circled

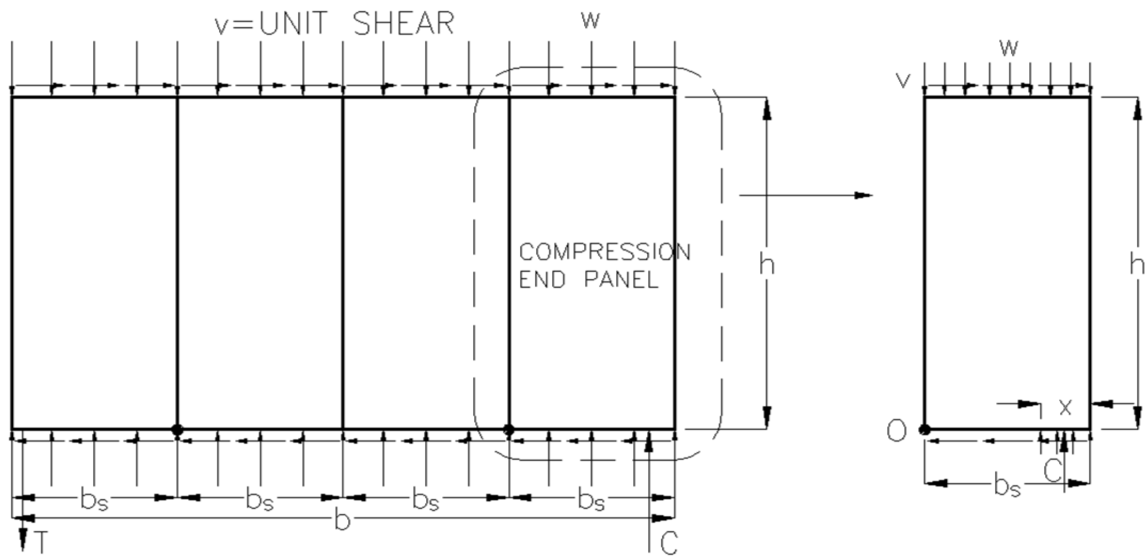


Figure 5.11: Free-body diaphragm of compression end panel, used to determine compression reaction force, C.

In addition to perpendicular to the grain compression, CLT wall panel resistance must be checked for axial loading. The axial resistance for this check shall be in accordance with NDS considering cross section dimension of the compression zone.

The free-body diagram for determination of the compression reaction force, C, in Figure 5.11 is used to ensure adequate compression bearing for the CLT shear wall end panel as well as a method to determine the axial loading for the CLT wall panel. While a centroid of the compression zone can be calculated, actual rotation of the end panel is about the compression toe of the end panel. It should be noted that summation of forces vertically for the free-body of the compression end panel depicted in Figure 5.11 might erroneously suggest a strengthening of the vertical joint connection to maintain stability due to location of the centroid of the compression zone away from the compression edge. Such added fastening of the vertical joint, based upon erroneous assumption of rotation about the centroid of the compression zone, is not to be provided at the vertical joint location because of potential to inhibit the intended rocking behavior.

5.2.4.3 Tension Force

The tension force required to maintain static equilibrium for overturning is based on as assumed rotation behavior of individual CLT panel within the shear wall as opposed to the shear wall overturning as monolithic rigid body. The resulting tension force will be greater than calculated assuming the full length of the multi-panel wall overturns as rigid body. From static equilibrium of the tension end panel (see Figure 5.12), controlling tension force is based on summing moment about O which is the compression edge of the tension end panel.

Tension force calculation by static equilibrium of tension end panel (see Figure 5.12):

$$\sum M_o = 0$$

Overturning moment due to factored loads
= (overturning moment due to unit shear, v, kip/ft)
+ (overturning moment due to dead load, w, kip/ft)

$$= -(\boldsymbol{v} * \boldsymbol{b_s} * \boldsymbol{h}) + (\mathbf{0.7} * \boldsymbol{w} * \boldsymbol{b_s} * \frac{\boldsymbol{b_s}}{\mathbf{2}}) \qquad \textbf{(5.8)}$$

where

v=unit shear, kip/ft
w= unit gravity including wall panel self-weight, kip/ft
0.7= load factor on D from LRFD combination 7 for assumed value of SDS= 1.0 (see above)
b_s= CLT panel length, ft
h= CLT panel height, ft

Resisting moment due to tension devise (assumed to be located at the panel's tension edge for purposes of this example):
= (Tension force, T, kip)*(moment arm, ft)

$$= \boldsymbol{T} * (\boldsymbol{b_s}) \qquad \textbf{(5.9)}$$

where

b_s= CLT panel length, ft. The actual moment arm will vary based on actual location of the tension device. For simplicity in this example, it is assumed that the tension device is located at the panel's edge and moment arm is equal to panel length, bs.

The equation for static equilibrium considering combined moment due to factored load and resisting moment due to tension device is:

$$\boldsymbol{T * b_s - (v * b_s * h) + 0.7 * w * b_s * \left(\frac{b_s}{2}\right) = 0} \quad \textbf{(5.10)}$$

Solving for tension force, T, used for sizing tension device (i.e. rod or holddown), gives the following:

$$\boldsymbol{T = v * h + 0.7 * w * \left(\frac{b_s}{2}\right)} \quad \textbf{(5.11)}$$

In the case of single panel configuration (see Figure 5.13) there is a slight change in moment arm values. This is due to lack of an opposing uplift from adjacent panel. Eq. 5.11 given below was used in lieu of Eq. 5.9.

$$\boldsymbol{T * \left(b_s - \frac{x}{2}\right) - v * b_s * h + 0.7 * w * b_s * \left(\frac{b_s}{2} - \frac{x}{2}\right) = 0} \quad \textbf{(5.12)}$$

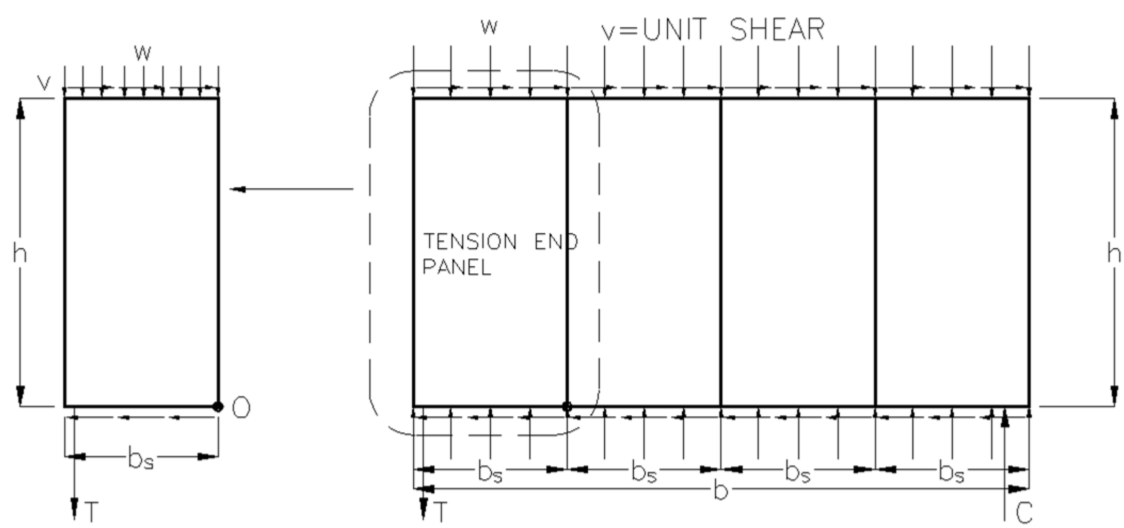


Figure 5.12: Free-body diaphragm of tension end panel in a multi-panel configuration, used to determine tension force, T.

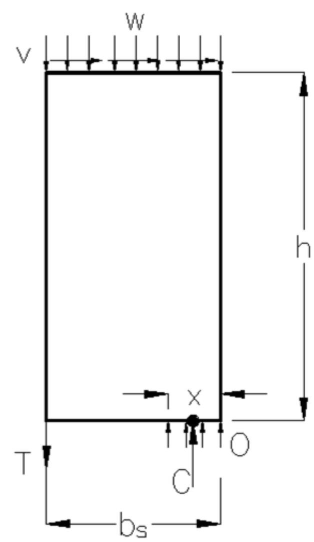


Figure 5.13: Free-body diaphragm of single panel configuration, used to determine tension force, T.

5.2.5 Deflection

Studies (Ceccotti et al, 2006; Dujic et al., 2004, 2005, 2006; Popovski et al., 2010; Gavric et al., 2015) have shown that two main sources of deformation in the CLT shear all are sliding and rocking. For the purpose of this study three primary components of deflection are incorporated in the shear wall deflection equation and these include: individual wall panel bending, sliding, rigid body overturning. Individual panel rocking is included for the cases having multi-panel configuration. The deflection method accounts for the difference in observed stiffness of single and multi-panel CLT walls tested as well as influence of individual panel aspect ratio on shear wall deflection. Components of shear wall deflection are shown in Figure 5.14.

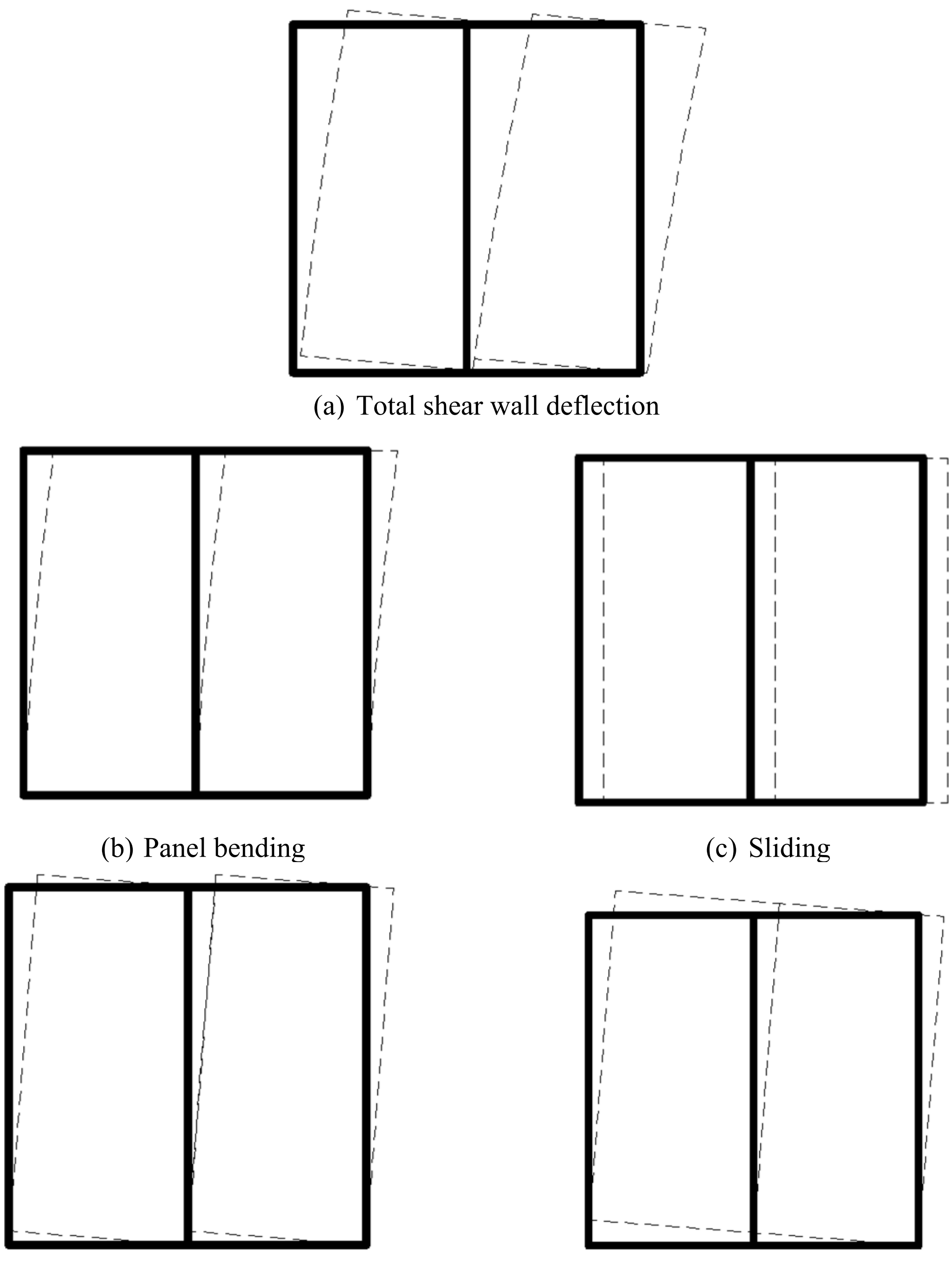


Figure 5.14: Shear Wall Deflection represting deflection components due to panel bending, sliding due to fastener slip, rotation due to fastener slip at vertical joints, and rigid body rotation

The following deflection equation is proposed for CLT special shear walls. The use of multiple terms to address different shear wall deflection components is similar to the deflection equation approach provided for light-frame wood structural shear walls in SDPWS (ANSI/AWC SDPWS, 2015). Shear wall deflection, δ_{SW}, shall be permitted to be calculated by use of the following equation:

$$\Delta = \frac{576 v b_s h^3}{EI_{eff}} + 3\Delta_{nail\ slip,h} + 2\,\Delta_{nail\ slip,v}\frac{h}{b_s} + \Delta_a \frac{h}{\sum b_s} \quad \textbf{(5.13)}$$

where:

v = induced unit shear, plf
EI_{eff} = Effective EI of CLT for in-plane bending, lb-in^2
h = CLT panel height, ft
b_s = individual CLT panel length, ft
$\sum b_s$ = sum of individual CLT panel lengths, ft
$\Delta_{nail\ slip,h} = \frac{V_{nail\ load}}{6700}$, in.
$\Delta_{nail\ slip,v} = \Delta_{nail\ slip,h}$, in. (= 0 for no vertical joints, i.e. single panel shear wall)
Vnail load = load per nail, lbf (calculated as total shear load at base of wall divided by total number of nails in base connectors)
Δ_a = total vertical elongation of wall anchorage system (including tension device fastener slip, elongation, compression zone deformation etc.) at the induced unit shear in the shear wall measured at the edge of the shear wall, in.

The bending term, $\frac{v b_s h^3}{3 * EI_{eff}}$, in the deflection equation is for the shear wall performing like a cantilevered beam. The term is simplified to $\frac{576 v b_s h^3}{EI_{eff}}$, to account for the unit conversion so that EI_eff can be in lb-in^2 and other units can be in feet, similar to SDPWS. EI_eff is the effective in-plane panel stiffness for bending to account for partial composite behavior between adjacent parallel laminations and it can be obtained from CLT manufacturer's literature or alternatively may be determined as follows:

EI_{eff} is calculated based on the following equation presented in Blass and Fellmoser (2004)

$$(EI)_{eff} = \left[1 - \left(1 - \frac{E_{90,T}}{E_{0,L}}\right)\frac{a_{m-2} - a_{m-4} + \cdots \pm a_1}{a_m}\right] E_{0,L} * \frac{bs^3 * a_m}{12}$$

where:
$E_{0,L}$= modulus of elasticity parallel to the grains for longitudinal layers
$E_{90,T}$= modulus of elasticity perpendicular to the grains for transverse layers
a_m= full thickness of the panel
In the case of a 5 layer panel, a_m, a_{m-2} and a_{m-4} are a_5, a_3 and a_1, respectively, and are shown in Figure 5.15.

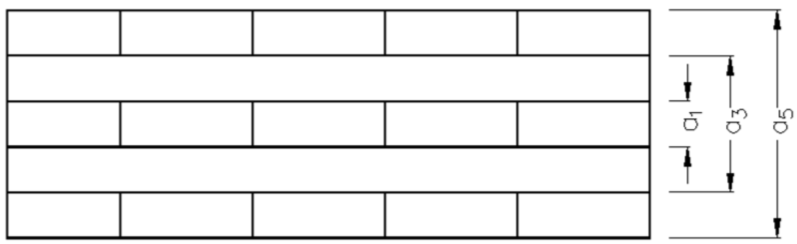


Figure 5.15: Layer thickness definition for a 5-layer panel

The method presented by Blass and Fellmoser (2004) is based on the composite theory where strength and stiffness properties for CLT are calculated for a fictitious homogenous cross section with grain of layers parallel to the stress direction. This is done through calculation of composition factors that are taken as the ratio of the strength or stiffness of the true cross section to the strength or stiffness of the said fictitious homogenous section. It is important to note that the method does not account for shear deformation.

The sliding term, $\frac{V_{nail\,load}}{135{,}000\,D^{1.5}}$, addresses sources of deformation in the connector that includes both nails and bolts. The slip constant takes into account the increased slip for loading perpendicular to the grain in the nailed connection. With the nail diameter of 0.135 in. used in the all the connectors in this study, allows the use of the simplified term $\frac{V_{nail\,load}}{6700}$. The deflection equation also explicitly breaks out sliding from multi-panel rotation due to vertical joint slip. If there is no vertical joint then vertical joint slip = 0. The final term in the deflection equation represents lateral deformation due to rigid body overturning assuming the point of rotation is at the compression toe.

5.3 CLT Special Shear Wall System

5.3.1 General Requirements

The proposed CLT system is intended for platform construction where all individual wall panels are single story clear height and bear on or support CLT floor panels. This precludes application of the proposed system for balloon type construction. The design method requires the same seismic detailing (i.e. panel aspect ratio and shear connectors) for all CLT walls whether part of the designated seismic force resisting system or not to promote deflection compatibility. These loads are then distributed to shear walls within the wall line based on stiffness of each wall within the wall line.

The design method specifically requires that all CLT wall panels, whether part of the designated seismic force resisting system or not, be included in the structural model to evaluate for presence of structural irregularity in accordance with ASCE 7-16 12.3.2. It is conservatively specified that wall panels not part of the designated seismic force resisting system, and therefore likely without required overturning restraint, be modeled assuming that such panels develop strength and stiffness associated with full overturning restraint provided at each end of each shear wall.

Connections occur in addition to those of the designated seismic force resisting system. These can include attachment of floor panels to wall panels below for out of plane loading, interconnection of walls at intersections, and use of conventional tie-down devices at wall ends. For these and other load path connections, it is required that fastener embedment is sufficient to promote Mode III or IV yielding and that design include evaluation of wood strength limits states of NDS Appendix E.

5.3.2 CLT Diaphragm

Fasteners used in diaphragm connections are required to have sufficient embedment to promote Mode III or IV yielding and that design include evaluation of wood strength limits states of NDS Appendix E. ASCE 7-16 Section 12.11 requirements are applicable to ensure wall-to-diaphragm integrity.

CHAPTER 6: DESIGN REQUIREMENTS FOR CLT SPECIAL SHEAR WALL SEISMIC FORCE RESISTING SYSTEMS

This chapter was developed in collaboration with Mr. Philip Line of the American Wood Council (AWC) and members of the project team. The discussion presented in this chapter is based on the material presented in the previous chapter and is intended to form design provisions for the CLT special shear wall system.

6.1 Scope

These provisions are intended for use in the design and construction of structural cross-laminated timber (CLT) walls and connections that are part of the seismic-force-resisting system. Capacity design principles are employed to ensure development of the expected shear capacity of the prescribed nailed connectors of the CLT special shear wall. The provisions provided herein shall be applied in combination with requirements of the 2015 *National Design Specification® for Wood Construction* (*NDS®*) including Appendix E, ASCE/SEI 7-16, *Minimum Design Loads for Buildings and Other Structures,* and the applicable building code.

6.1 CLT Special Shear Walls System Requirements

The system of construction for the CLT seismic force resisting system shall comply with all of the following:

a) Platform frame construction whereby CLT floor panels bear on and are supported by CLT walls below.
b) CLT walls shall be composed of one or more CLT wall panels.
c) CLT wall panels shall be solid over the height of the panel and have a height to length ratio that is not less than 2:1 and not greater than 4:1.

d) CLT walls shall be classified as either (1) part of the designated seismic force resisting system (CLT special shear walls) or (2) not part of the designated seismic force resisting system.

e) CLT special shear walls shall be configured to resist lateral seismic forces using “stacked shear wall” construction of CLT special shear walls of the same length and plan location at each story.

f) CLT wall panels not considered part of the designated seismic force resisting system shall utilize construction details including CLT wall panel aspect ratio limits and prescribed connectors associated with the CLT special shear walls except that a tension device for overturning at each end of each shear wall shall not be required.

g) CLT wall panels not considered to be part of the designated seismic force resisting system 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 seismic force resisting system. The design shall provide for the effect of these elements on the structural system at structural deformations corresponding to the design story drift and additionally, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in accordance with 12.3.2 of ASCE 7 based on their placement, strength and stiffness. For evaluation of presence of structural irregularity, CLT wall panels that are not part of the designated seismic force resisting system shall be modeled assuming they develop in-plane shear strength and stiffness associated with CLT special shear walls of the same construction. Where placement of these elements produces an out-of-plane offset or in-plane discontinuity irregularity, a designed load path for overturning induced compression forces shall be provided in accordance with ASCE 7 Sec. 12.3.3.3.

h) Unless the diaphragm can be idealized as flexible in accordance with ASCE 7 Section 12.3.1, distribution of seismic forces shall utilize either semi-rigid diaphragm modeling or the idealized as rigid diaphragm assumption.

i) Load path connections not specifically prescribed as part of the designated seismic force resisting system shall have sufficient embedment to develop Mode III or Mode IV yielding, and as applicable, comply with net section tension rupture, row tear-out, group tear-out in accordance with NDS Appendix E.

6.2 CLT Special Shear Walls

CLT special shear wall detailing requirements intend to promote yielding of nails and metal connectors at CLT panel edges to enable combined rocking and sliding behavior of individual wall panels prior to occurrence of the ultimate shear wall strength limit state. Special detailing requirements include:

a) CLT panels of prescribed aspect ratios,
b) prescribed nailed connectors at bottom of panel, top of panel, and vertical joints of multi-panel shear walls,
c) minimum required capacity for overturning device, and
d) compression zone length requirements for overturning resistance.

6.2.1 Unit Shear Capacities

The nominal unit shear capacity, v_s, of CLT special shear walls shall be in accordance with Table 6.1 and all of the following requirements:

a) CLT moisture content shall be 16% or less and specific gravity, G, shall be 0.42 or greater.

b) CLT forming either a single panel or multi-panel shear wall shall not have aspect ratio, h/b_s, greater than 4:1 nor less than 2:1, where h = wall panel height and b_s = wall panel length. Individual panels forming the multi-panel shear wall shall have the same aspect ratio, h/b_s.

c) Connectors shall be spaced at the same average on-center spacing at wall panel top, bottom, and vertical edges. Bottom and top of wall panel connections shall extend to within 12 in. of each end of each panel of a single or multi-panel shear wall. Each panel shall have at least two shear connectors.

d) Minimum panel thickness in accordance with Table 6.1 shall be doubled where connectors on opposite sides of the CLT panel are directly opposed. Alternatively, connectors on opposite faces of the panel shall be offset such that that connector nails from opposing sides do not overlap.

The LRFD factored unit shear resistance shall be determined by multiplying the nominal unit shear capacity by a resistance factor, ϕ_D, of 0.50. The ASD allowable unit shear capacity shall be determined by dividing the nominal unit shear capacity by the ASD reduction factor of 2.8. Nominal unit shear capacity, v_s, in accordance Table 6.1 is based on the number of connectors on one side of the wall panel. Where both sides are provided with connectors, the nominal unit shear capacity shall be permitted to be taken as the sum of the nominal unit shear capacities of each side. Nominal unit shear capacity of one side is determined as the number of bottom of wall connectors on one side, NC, multiplied by connector capacity divided by the panel length, b_s, in feet.

Table 6.1. Nominal unit shear capacity of CLT special shear walls, plf

Base and top of wall connection	Vertical joint connection	Minimum panel thickness, inch	Nominal unit shear capacity, v_s, plf
Type A (see Table 6.2)	Type E (see Table 6.3)	3.5	$v_s = NC^*(2605/b_s)$ **(6.1)**
Type B (see Table 6.2)	Type F (see Table 6.3)	3.5	$v_s = NC^*(5210/b_s)$ **(6.2)**

6.2.2 Overturning resistance

6.2.2.1 The load path for overturning resistance shall have required design capacity in accordance with basic load combinations of ASCE 7 Section 12.4.2.3 (load combinations without overstrength) and comply with additional requirements of 6.3.2.2 and 6.3.2.3.

6.2.2.2 Each end of each shear wall shall be provided with a tie-down designed to transfer the overturning induced tension forces. Tide-down devices shall comply with the following:

a) Where continuous tie-down devices are used, rods at each level are designed for cumulative overturning tensile forces and bearing shall be provided at floor level above each story. Tie-down rod elongation shall not exceed 0.18 in per story using Allowable Stress Design (ASD).

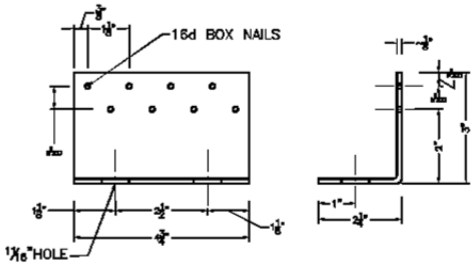
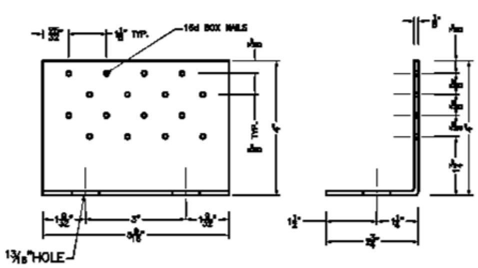
b) The nominal strength of the tie-down device shall not be less than required to resist the net uplift forces associated with development of the maximum expected shear wall unit shear capacity, where expected shear wall unit shear capacity is taken as 1.15 times the nominal unit shear capacity in accordance with Table 6.1. The nominal strength of the tie-down device shall be taken as the smaller of: i) required test strength of the tie-down device in accordance with manufacturer's literature, ii) calculated nominal strength of the device in accordance with applicable steel standards, and iii) calculated nominal strengths associated with connection wood strength limit states of net section tension rupture, row tear out, and group tear out in accordance with NDS Appendix E.

6.2.2.3 Each end of each shear wall shall be provided with a designed compression load path. The compression zone for overturning induced compression forces shall be contained within the outermost wall panel for multi-panel shear walls based on an assumed uniform distribution of bearing stress. CLT wall panel resistance to induced axial compression forces shall be determined using cross section dimensions associated with the compression zone.

6.2.3 In-plane shear at top and bottom of shear wall

The connection of the shear wall for shear transfer at the top and bottom of the shear wall shall be in accordance with Table 6.2. Connector spacing and placement within a wall panel shall be in accordance with 6.3.1.

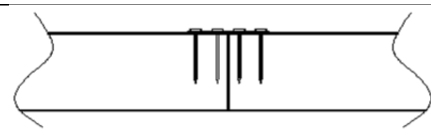
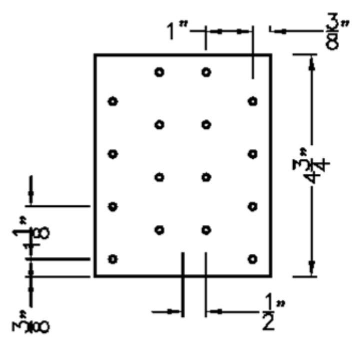
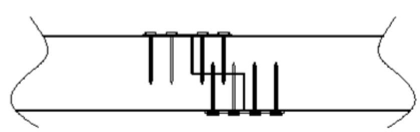
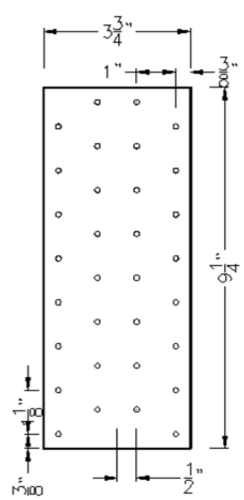
Table 6.2. Connection for shear transfer at the top and bottom of the shear wall

Connector type	Connector details	Vertical leg fasteners	Horizontal leg fasteners
Type A: 0.108 in. x 2.25 in. x 3 in. steel angle		(8) 16d box nails, 3-1/2 in. x 0.135 in. diameter	(2) 5/8 in. diameter bolts x 4-1/2 in. long (minimum) or, (2) 5/8 in. full-body diameter lag screws with 2.75 in. thread penetration (minimum) excluding tapered tip and 5D unthreaded shank length (minimum)
Type B: 0.108 in. x 2.75 in. x 4 in. steel angle		(16) 16d box nails, 3-1/2 in. x 0.135 in. diameter	(2) 3/4 in. diameter bolts x 4-1/2 in. long (minimum) or, (2) 3/4 in. full-body diameter lag screws with 4.75 in. thread penetration (minimum) excluding tapered tip and 5D unthreaded shank length (minimum)

6.2.4 Shear transfer at vertical joint in the multi-panel shear wall

The connection of vertical joints of a multi-panel shear wall shall be in accordance with Table 35. Connector spacing and placement within a wall panel shall be in accordance with 6.3.1.

Table 6.3. Connection for shear transfer at vertical joint in multi-panel shear wall

Connector type	**Connector details**	**Fasteners on each side of vertical joint**
Type E: 0.105 in. ASTM A653 Grade 33 steel plate	 	(8) 16d box nails, 3-1/2 in. x 0.135 in. diameter
Type F: 0.105 in. ASTM A653 Grade 33 steel plate		(16) 16d box nails, 3-1/2 in. x 0.135 in. diameter
		

6.2.5 Shear wall deflection

Shear wall deflection, δ_{SW}, shall be permitted to be calculated by use of the following equation:

$$\Delta = \frac{576 v b_s h^3}{EI_{eff}} + 3\Delta_{nail\ slip,h} + 2\,\Delta_{nail\ slip,v}\frac{h}{b_s} + \Delta_a \frac{h}{\sum b_s} \qquad \textbf{(6.3)}$$

where:

v = induced unit shear, plf
EI_{eff} = Effective EI of CLT for in-plane bending, lb-in^2
h = CLT panel height, ft
b_s = individual CLT panel length, ft
$\sum b_s$ = sum of individual CLT panel lengths, ft
$\Delta_{nail\ slip,h} = \frac{V_{nail\ load}}{6700}$, in.
$\Delta_{nail\ slip,v} = \Delta_{nail\ slip,h}$, in. (= 0 for no vertical joints, i.e. single panel shear wall)
Vnail load = load per nail, lbf (calculated as total shear load at base of wall divided by total number of nails in base connectors)
Δ_a = total vertical elongation of wall anchorage system (including tension device fastener slip, elongation, compression zone deformation etc.) at the induced unit shear in the shear wall measured at the edge of the shear wall, in.

6.2.6 Distribution of shear

Shear distribution to individual shear walls in a wall line shall provide the same calculated deflection in each shear wall.

6.3 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 III 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 is provided in Table 6.4. Typical detailing is provided in Appendix D, and example design calculations for an archetype for R=3 are attached in Appendix F. All archetype designs and the corresponding modeling henceforth are performed with CLT shear walls that used A3 type connectors. Also while all the archetypes were checked for R=3, only archetypes with high aspect ratio panel configurations were checked for R=4. This was due to the relatively better deformation capacity demonstrated by the high aspect ratio panel configurations during the shear wall testing.

Table 6.4. Summary of CLT Isolated Shear Wall Testing Results

				Strength Design						Drift Design		
				Fmax (kip)								
Test #	Length (ft)	No. connectors	Connector type	+ve	-ve	LRFD Nominal Capacity	LRFD Design Strength	Test/LRFD Design Strength*	Load in test wall (lb)	Min. Measured Total Deflection (in.)	Theoretical deflection (in.)	Measured/Theory
03	4	3	A3	14.79	14.62	7.8	3.91	3.74	3907.5	0.48	0.60	0.80
04	4	3	A3	15.7	14.22	7.8	3.91	3.64	3907.5	0.50	0.56	0.89
05	4	3	A3	17.9	18.15	7.8	3.91	4.58	3907.5	0.36	0.52	0.69
06*	4	3	A3	19.51	18.02	7.8	3.91	4.61	3907.5	0.51	0.52	0.98
09	4	3	A3	15.11	11.62	7.8	3.91	2.97	3907.5	0.87	0.64	1.36
10	4	4	A3	15.2	13.12	10.4	5.21	2.52	5210.0	0.81	0.75	1.08
11	4	2	A3	8.03	8.7	5.2	2.61	3.08	2605.0	0.63	0.68	0.93
13	4	2	A3	7.597	10.72	5.2	2.61	2.91	2605.0	0.68	0.61	1.11
14	4	3	A3	19.98	16.52	7.8	3.91	4.23	3907.5	0.51	0.57	0.89
15	4	2	A3	10.16	11.02	5.2	2.61	3.89	2605.0	0.58	0.61	0.95
17	4	4	A3	24.5	21.9	10.4	5.21	4.20	5210.0	0.64	0.64	1.00
18	4	2	A3	6.74	7.3	5.2	2.61	2.58	2605.0	0.67	0.70	0.96
19	4	5	A3	17.93	16.43	13.0	6.51	2.52	6512.5	0.78	0.74	1.05
20	4	5	A3	19.11	18.69	13.0	6.51	2.87	6512.5	0.61	0.67	0.91
21	2	2	A3	7.19	5.95	5.2	2.61	2.28	2605.0	1.55	1.75	0.89
23	2 (2)	4	A3	13.34	13.84	10.4	5.21	2.56	5210.0	0.84	1.01	0.83
24	4	2	B3	18.58	18.62	10.4	5.21	3.57	10420.0	0.44	0.69	0.64
25	4	3	B3	27.8	29.24	15.6	7.82	3.56	5210.0	0.74	0.63	1.17
26	4 (2)	8	A3	23.72	24.3	20.8	10.42	2.28	7815.0	0.67	0.66	1.02
											Avg.	0.952

* The ratio was calculated based on the maximum of the positive and negative excursion

CHAPTER 7: NUMERICAL MODEL AND ANALYSES

According to FEMA P695 (2009) the nonlinear numerical models should simulate all significant deterioration mechanisms that can lead to collapse i.e. degradation in stiffness and strength, and inelastic deformation. Analytical models used to simulate CLT behavior at the component and at the assembly level is explained in this chapter.

7.1 CLT Connections

As observed in the testing, similar to light-frame wood shear walls, CLT connector and wall behavior is governed by nail behavior and this force-displacement response is highly nonlinear. Various numerical models which vary in terms of detail and complexity have been proposed to predict hysteretic behavior of fasteners in wood.

Lee (1987) used the model proposed by Polensek and Laursen (1984) to perform dynamic analysis on wood walls and diaphragms. The model uses a tri-linear backbone curve where hysteresis oscillates between control points in the positive and negative excursions and these points are obtained by performing a fit to the test data. Some important parameters in this model include type and size of the nail and wood material properties. Chou (1987) performed an experimental investigation of nailed wood connections which led to the development of a system nonlinear Kelvin models in series and in parallel. The proposed model was a modification of beam on elastic foundation analysis.

Stewart (1987) proposed a similar model whereby the backbone is defined by a tri-linear function and the proposed model considers degradation in strength and stiffness as well as the pinching behavior. Dolan (1989) used exponential functions to define the four loading and reloading paths and the proposed model was used in performing dynamic analysis on wood structures. These are shown in Figure 7.1 and 7.2, respectively.

Foliente (1995) proposed a hysteretic model for wood joints and structural system based on the modification of the Bouc-Wen-Baber-Noori (BWBN) hysteretic model for steel and concrete structures. The model considered degradation in stiffness and strength, and pinching. Foschi (2000) developed a hysteretic model for mechanical connections in wood where the connector is defined as an elasto-plastic beam in a medium which acts only in compression. This allows formation of the gaps between the nail and the wood and the pinching behavior is reflected in the hysteresis.

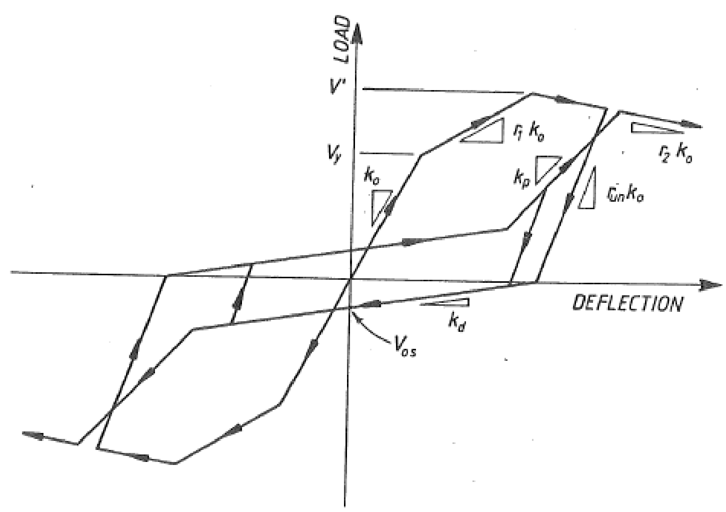


Figure 7.1: Numerical model for nailed wood frame connections (excerpted from Stewart (1987))

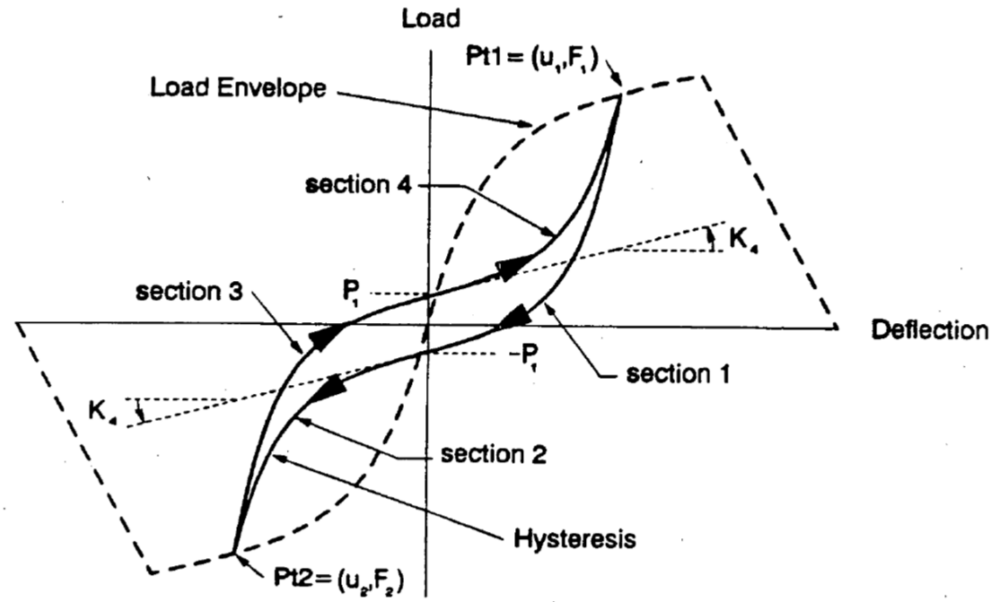


Figure 7.2. Numerical model for nailed wood frame connections (excerpted from Dolan (1989))

The 10-parameter hysteretic model, referred to as CUREE-SAWS model, developed by Folz and Filiatrault (2001) is perhaps the most widely used model since its development. It was developed as part of the CUREE-Caltech wood frame project and used in the Cyclic Analysis of Wood Shear Walls (CASHEW) program. A generic 10-parameter hysteretic model is shown in Figure 7.3. A reverse calibration procedure is used to obtain parameters for the connector. This type of reverse calibration has already been performed for various connection types and presented in the US CLT handbook (Karacabeyli and Douglas, 2013).

A number of studies (Shen et al., 2013; Schneider et al., 2015) have utilized the SAWS model to characterize CLT connector behavior and European studies (e.g. Pozza et al., 2017; Izzi et al., 2018) have also used similar models. As indicated earlier in the Chapter 4 of this dissertation, connection tests were performed on generic connectors and their behavior investigated as part of individual connector tests as well as within the wall tests. Test results are used to determine hysteretic parameters for the connector to facilitate application of the component equivalency method.

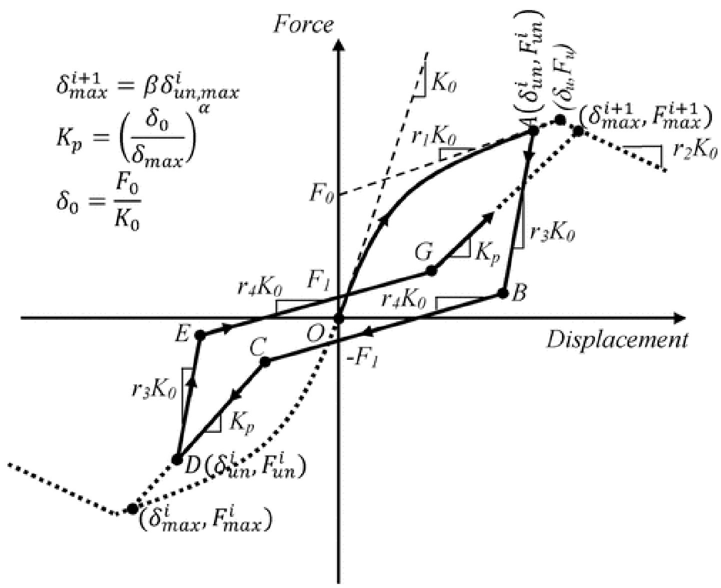


Figure 7.3. Loading Paths and Parameters (Folz and Filiatrault, 2001)

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

Parameter Characteristic

K_0 Initial stiffness

F_0 Resistance force parameter of the backbone

F_1 Pinching residual resistance force

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

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

r_3 Ratio of the unloading path stiffness to K_0

r_4 Ratio of the pinching load path stiffness to K_0

Δ_u Drift corresponding to the maximum restoring force of the backbone curve

α Stiffness degradation parameter

β Strength degradation parameter

7.2 CLT Wall Modeling

According to the 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 the primary energy dissipation mechanism exhibit hysteretic behavior similar to light-frame wood shear walls. The phenomenological CUREE-SAWS hysteretic model developed by Folz and Filiatrault (2001) as part of CUREE-Caltech (Consortium of Universities for Research in Earthquake Engineering) project is used in this study to characterize the CLT shear wall behavior. The model requires ten parameters to define force, stiffness, and their degradation as part of the hysteretic behavior. The 10-parameter hysteretic model was shown in Figure 7.3.

The 10-parameter hysteretic model used as part of this project is calibrated using the test results. To account for the difference in panel height between the model (10 ft) and the tests (8 ft) while preserving the aspect ratio, the load-displacement test data was adjusted 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. Under this adjustment, using a panel length of 5' in designs preserves the 2:1 aspect ratio and using a panel length of 2.5' in designs preserves the 4:1 aspect ratio for an assumed 10' wall height. The fitting was performed using the curve fitting tool in SAPWood (Pei and van de Lindt, 2007) and the parameters were determined considering the average of the positive and the negative envelope curve. The CLT Test matrix is provided in Table 7.1 and the CUREE-SAWS model parameters fit to the scaled test data are shown in Figure 7.4-7.10 and provided in Table 7.2. As mentioned at the end of Chapter 6, all the archetype designs and modeling are performed with A3 type connectors.

Table 7.1. CLT wall test matrix

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)	Holddown rod (in.)
01*,**,***	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
02*,**,***	V2	8	4	5	3.9	3	A1	0.68	(2) 5/8
03	V2	8	4	5	6.65	3	A3	0.68	(2) 5/8
04	V2	8	4	5	6.65	3	A3	1.28	(2) 5/8
05	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
06*	E1	8	4	5	6.89	3	A3	0.68	(2) 5/8
09	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****	V2	8	8	3	3.9	4	A3	-	(1) 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
27*****	E1	8	4	5	6.89	3	B3(3/16 in.)	-	(2) 3/4
28	E1	8	4	5	6.89	2	B3 (10 gauge)	-	(2) 5/8

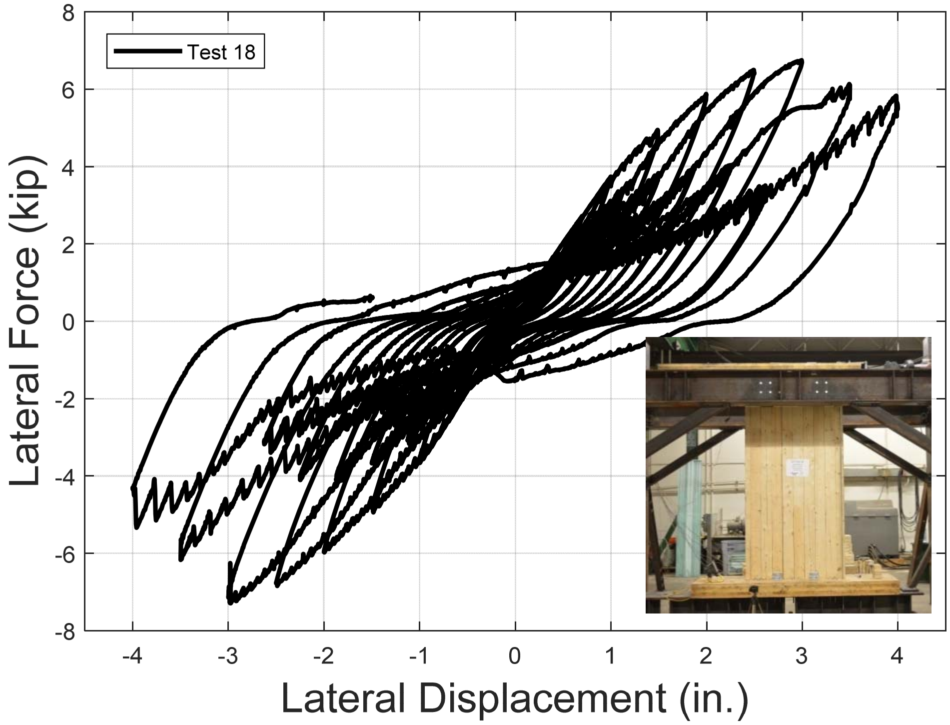
*Test 01,02, and 06 were performed with the imposed boundary condition. The imposed boundary condition is explained in detail in Section 4.2.3.3

** Test 01 and 02 were performed during the exploratory phase of the A type connector thickness

*** The top and bottom CLT panels matched the CLT wall panel grade in all testing, except for Test 01 and 02 where the wall panels were of V2 grade while the top and bottom CLT panels were of E1 grade. Top and bottom CLT panels of E1 and V2 grades were 6.89 in. and 6.65 in. in thickness, respectively.

**** Test 22 was (8ft x 8ft) 1:1 aspect ratio wall that is not covered by the design methodology

***** Test 27 and 28 were performed during exploratory phase of B type connector thickness



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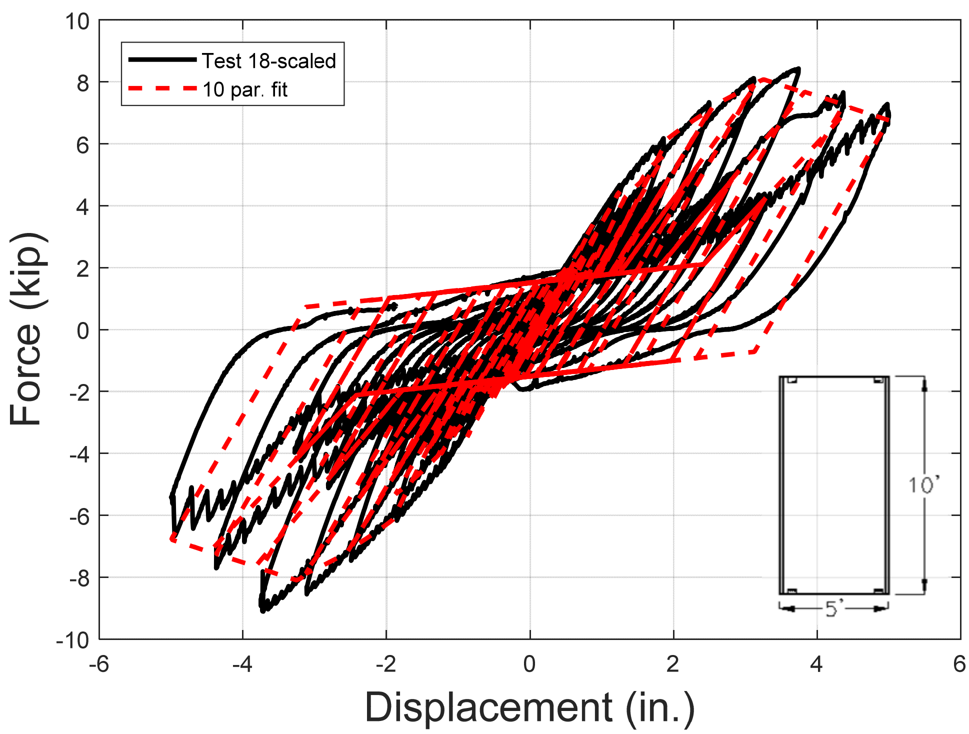
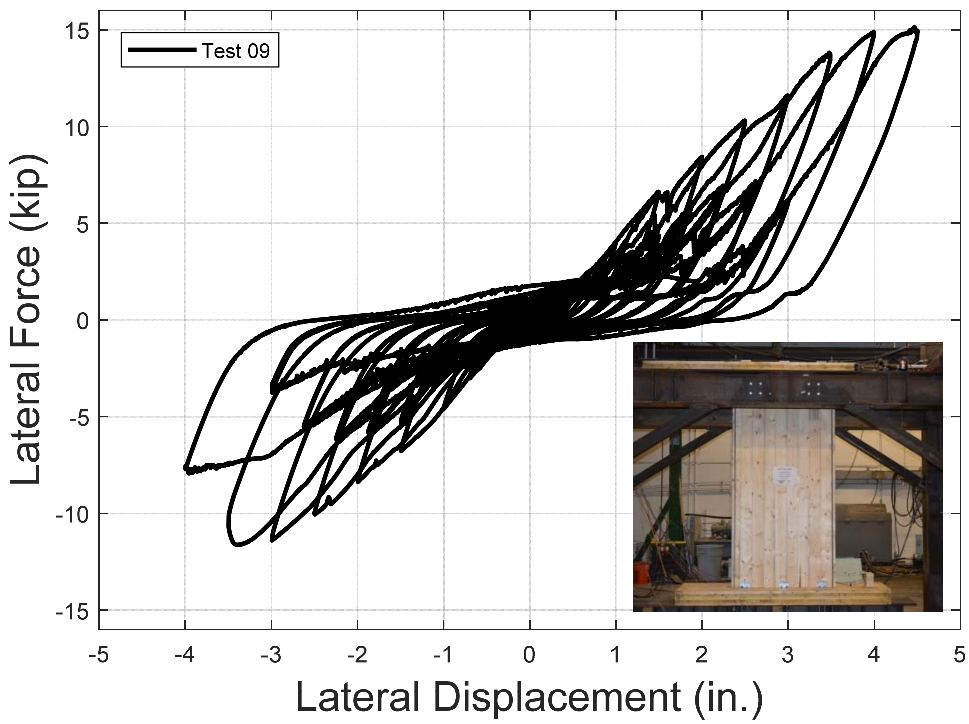
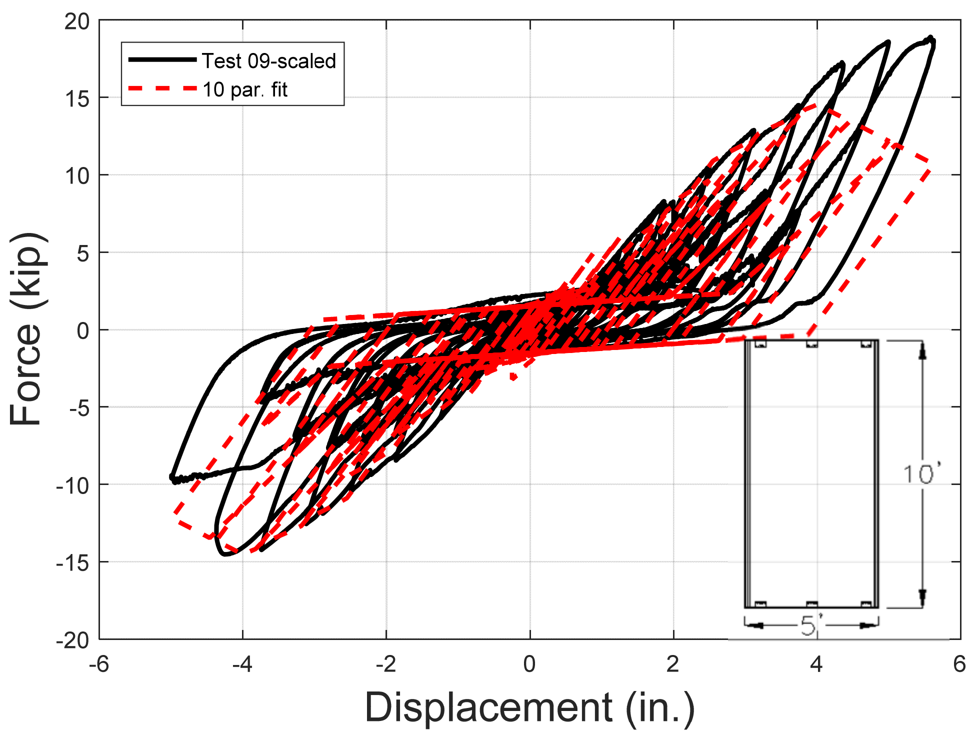


(b)

Figure 7.4. 2:1 aspect ratio panel, 2 connectors
(a) Test 18, 4ft x 8ft x 3.9 in. (b) Test 18 scaled data and hysteretic fit

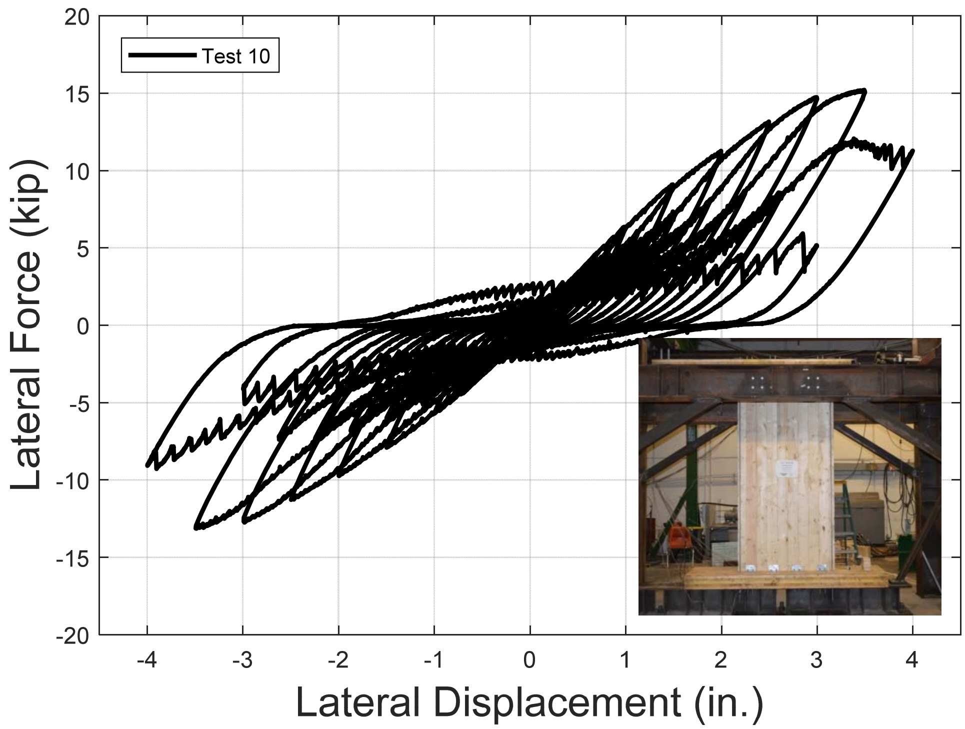


(a)

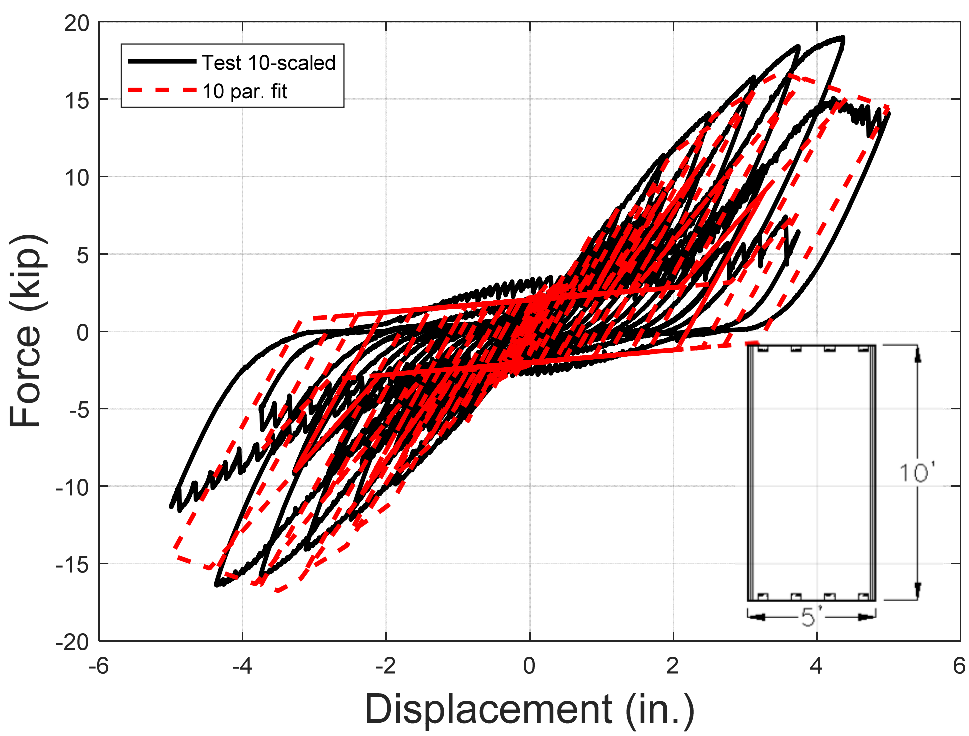


(b)

Figure 7.5. 2:1 aspect ratio panel, 3 connectors
(a) Test 09, 4ft x 8ft x 6.65 in. (b) Test 09 scaled data and hysteretic fit

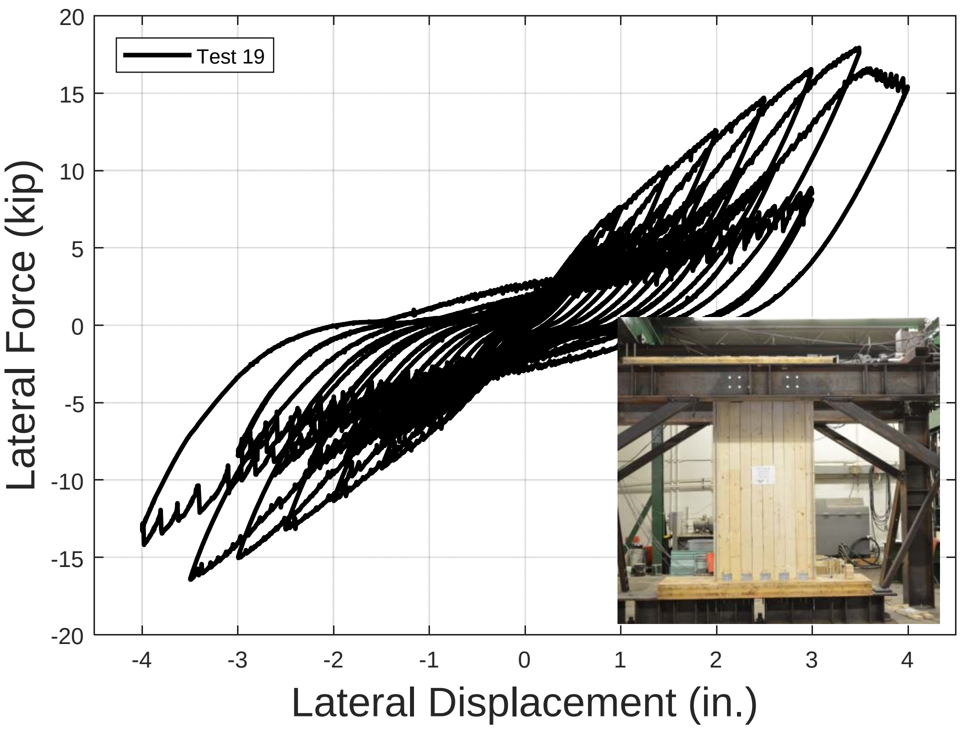


(a)



(b)

Figure 7.6. 2:1 aspect ratio panel, 4 connectors
(a) Test 10, 4ft x 8ft x 3.9 in. (b) Test 10 scaled data and hysteretic fit



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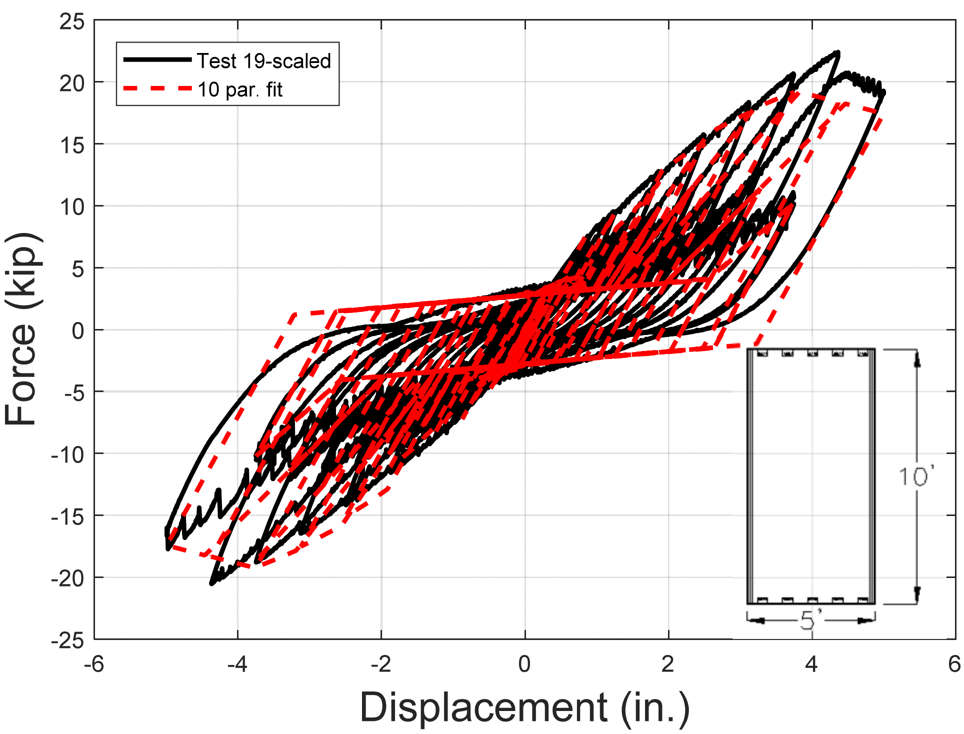

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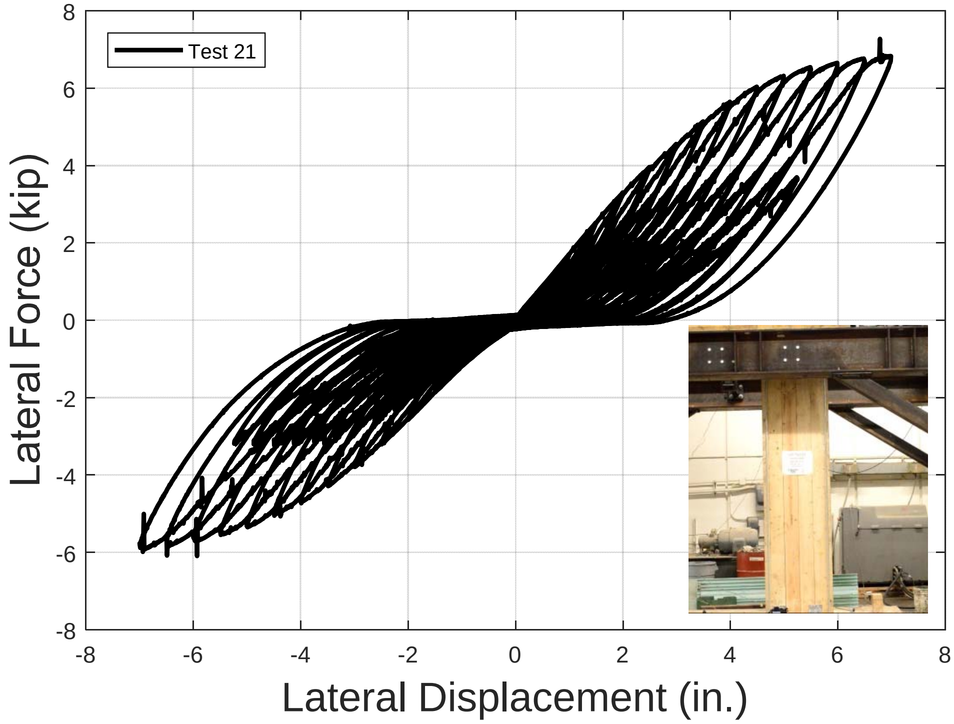
Figure 7.7. 2:1 aspect ratio panel, 5 connectors
(a) Test 19, 4ft x 8ft x 3.9 in. (b) Test 19 scaled data and hysteretic fit



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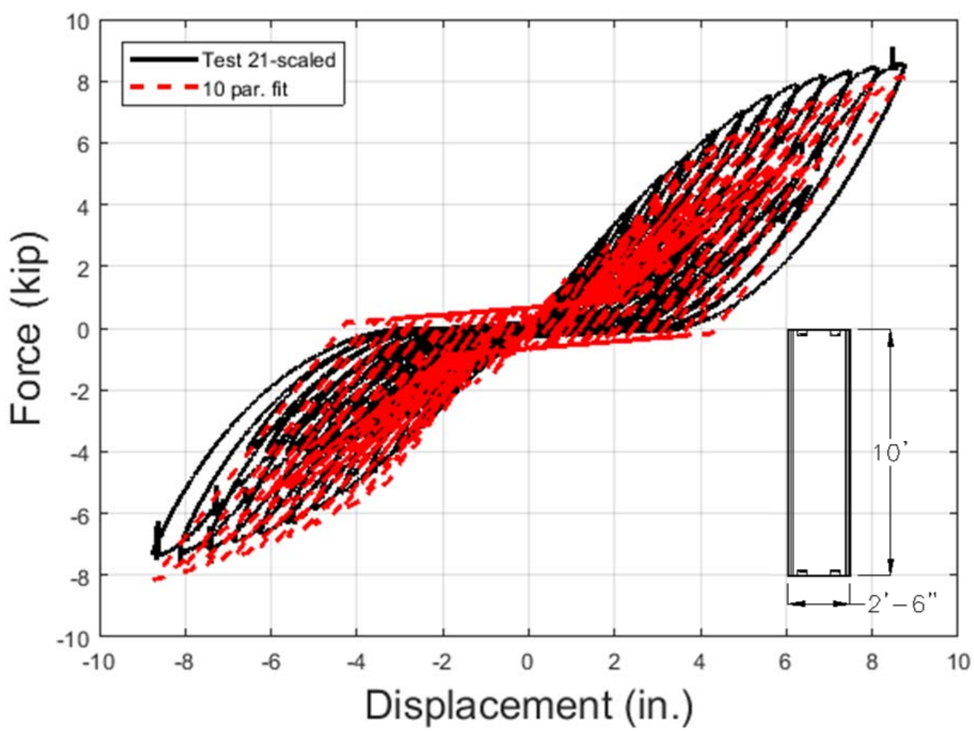
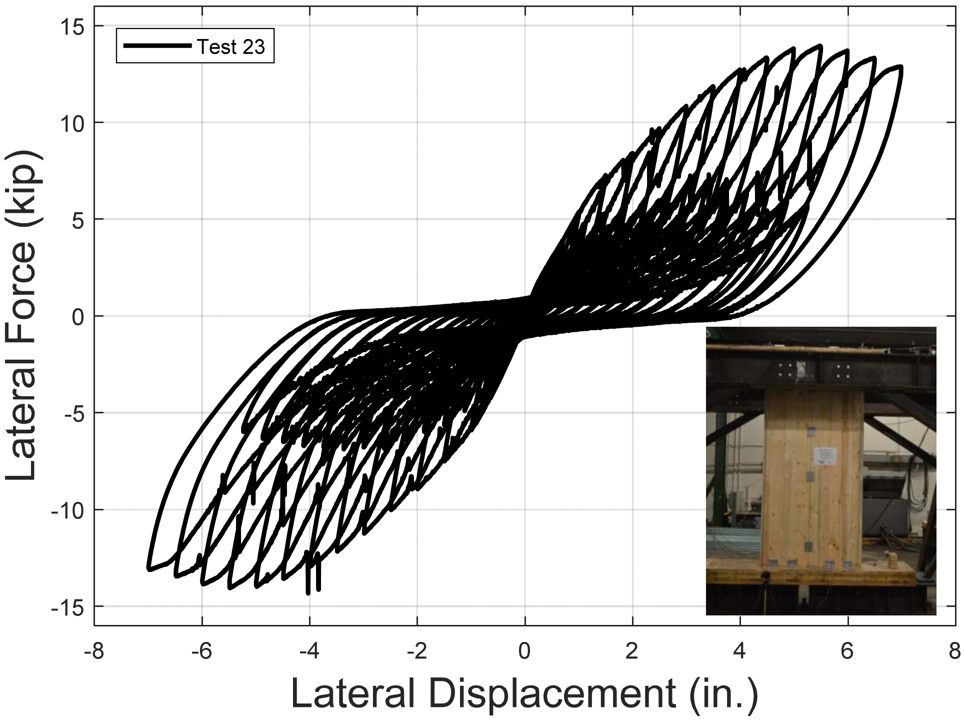
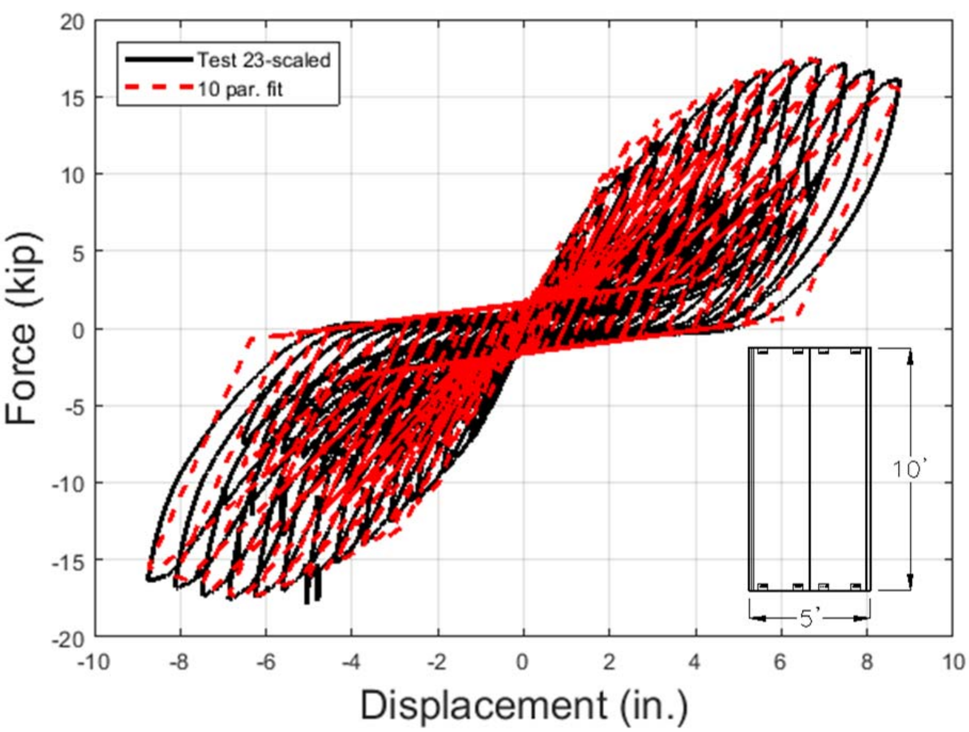


(b)

Figure 7.8. (1) 4:1 aspect ratio panels, 2 connectors/panel
(a) Test 21, (1)2ft x 8ft x 3.9 in. (b) Test 21 scaled data and hysteretic fit

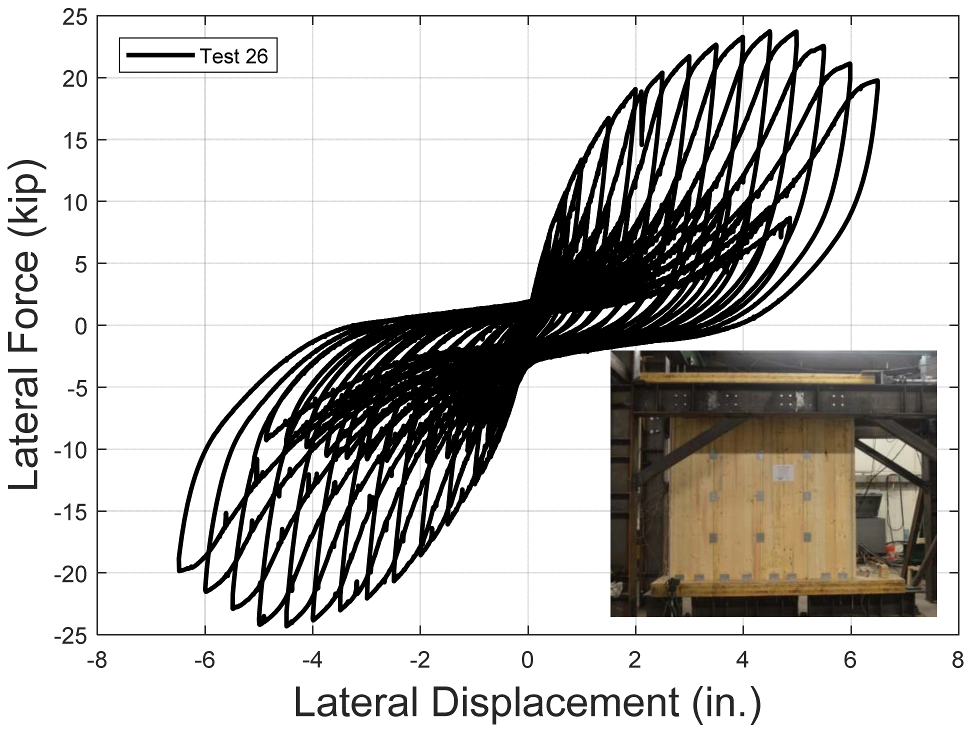


(a)

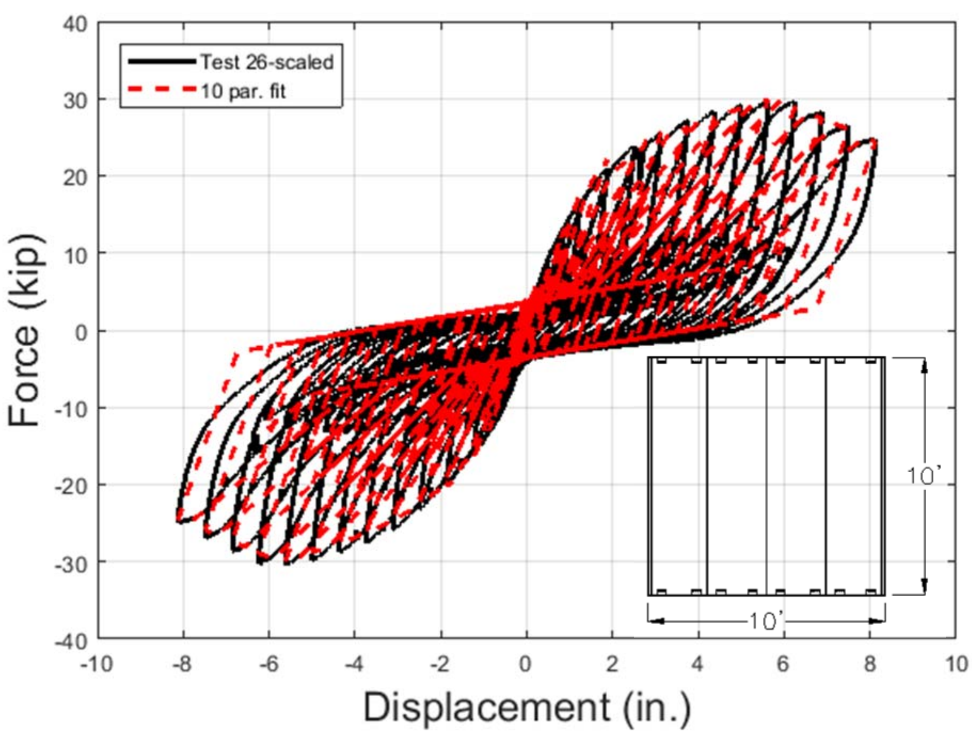


(b)

Figure 7.9. (2) 4:1 aspect ratio panels, 2 connectors/panel
(a) Test 23, (2)2ft x 8ft x 6.65 in. (b) Test 23 scaled data and hysteretic fit



(a)



(b)

Figure 7.10. (4) 4:1 aspect ratio panels, 2 connectors/panel (a) Test 26, (4)2ft x 8ft x 6.65 in. (b) Test 26 scaled data and hysteretic fit

Table 7.2. CUREE-SAWS parameters used for analysis

Configuration														
Panel length (ft)	No. of panels	Connector Type	No. of connector 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	5000	8760	1500	0.05	-0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	S	6000	16700	1500	0.1	-0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	S	8000	17670	2000	0.125	-0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	S	10000	21460	2750	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
2.5	1	A3	2	S	2250	7250	650	0.075	-0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	2	S	7500	14500	1600	0.075	-0.125	0.825	0.05	6.5	0.75	1.05
2.5	4	A3	2	S	19000	22000	3500	0.075	-0.125	0.825	0.05	5.75	0.75	1.05

Since wall lengths vary in a building, in the case multi panel configurations comprised of 4:1 aspect ratio panels, results from Test 21 (1 panel), Test 23 (two panels), and Test 26 (four panels) were used to scale the modeling parameters for varying wall lengths. This approach is similar to the light-frame wood shear wall modeling approach where for a certain unit shear strength configuration K_o, F_o, and F_I parameters are scaled in proportion to wall length while the other parameters remain unchanged (Koliou et al., 2018). In the case of CLT Δ_u values decreased based on the number of panels tested, e.g. looking at Table 7.2 these are 8.75 in., 6.5 in. and 5.75 in. for one, two and four multi-panel configurations, respectively. However, this trend was not a linear trend and based on the rocking behavior of individual CLT wall panels, Δ_u for longer walls (greater than 4 panels) is taken as 5.75 in. For example, given K_o, F_o, and F_I parameters for multi-panel configurations in Table 7.2, parameters for the multi-panel configuration comprised of five 4:1 aspect ratio panels are calculated as shown below in Figure 7.11.

Parameters for the multi-panel 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 multi-panel configuration comprised of 2:1 aspect ratio panel. Parameters for all different configurations are provided in Appendix E.

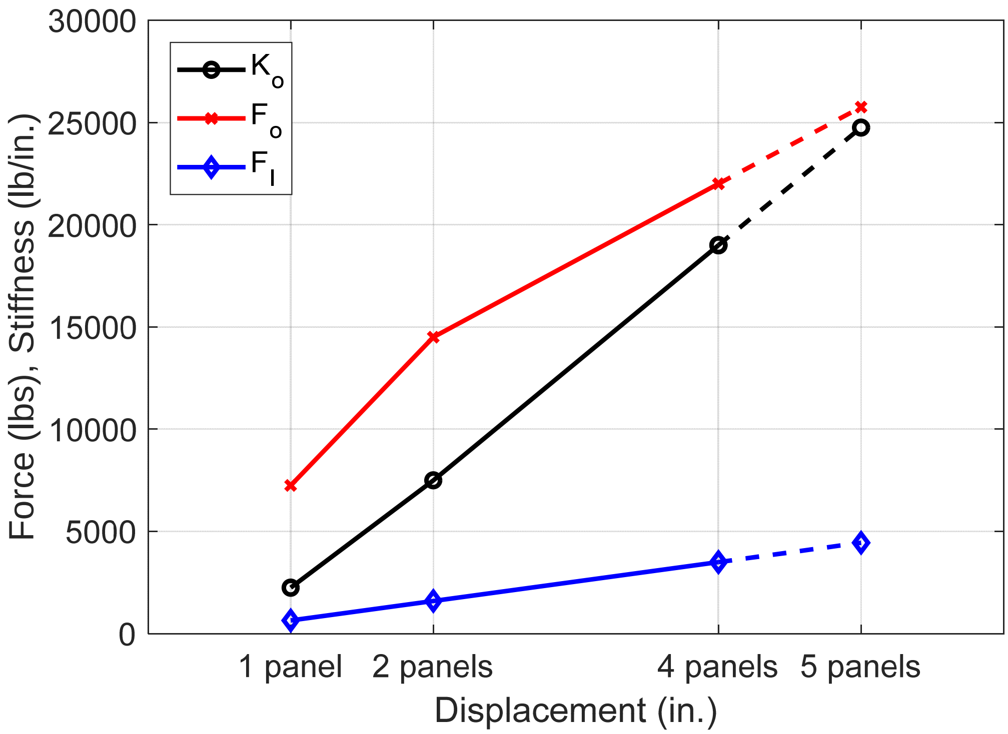


Figure 7.11. 4:1 aspect ratio multi-panel configuration parameter scaling to maintain unit shear strength and stiffness for multi-panel shear walls

7.3 Building System Modeling

CLT modeling can be performed with various levels of complexity and while a finite element (FE) based formulation can be used in certain cases, the computational effort for a study such as FEMA P695 typically demands utilizing models with simplified kinematic assumptions. Analysis will be performed using the SAPWood software (Pei and van de Lindt, 2007) that was developed as part of the NEESWood project for analysis of light-frame wood buildings. The software is based on the shear-bending coupled model and the assumptions involved in this model are as follows:

- Floor diaphragm as rigid plates having 6 degrees of freedom.
- Shear resistance can be represented using hysteretic springs.
- Overturning restraint can be represented using multi-linear springs.

- Effect of finish materials is ignored in this study

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 et al, 2010).

The kinematic model used in SAPWood is shown in Figure 7.12 and a simplified representation of the analytical model is shown in Figure 7.13. Buildings in SAPWood are considered to be composed of rigid diaphragms attached to shear walls that are represented by nonlinear springs and components that resist uplift are represented by tie-down springs. The degradation in strength and stiffness is captured through the 10-parameter hysteretic nonlinear spring model.

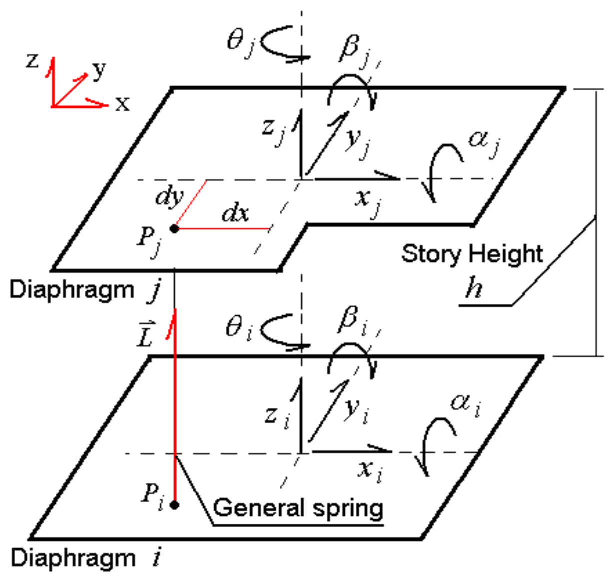


Figure 7.12. SAPWood kinematic model for nonlinear history analysis (Pei and van de Lindt, 2007)

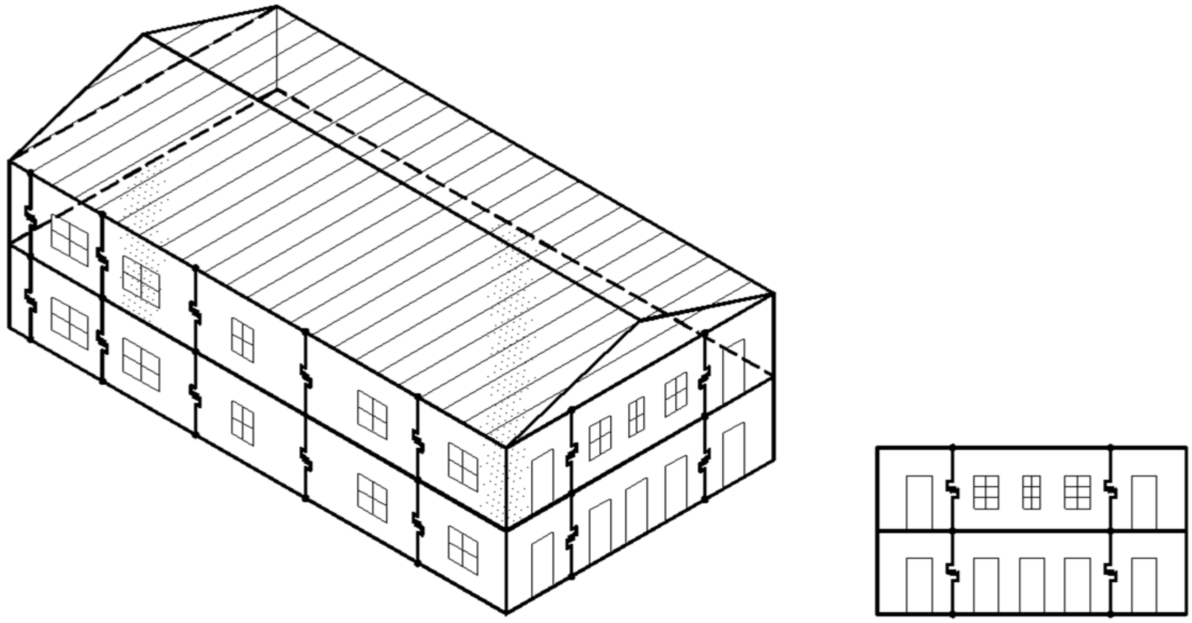


Figure 7.13. Analysis model and simplified 2D model

Once the designs were performed for all the archetypes, SAPWood was used to assemble system level structural model to carry out the nonlinear time history analysis.

7.4 Analysis

Analyses consisting of nonlinear static and dynamic analysis were performed on the index archetypes that met all requirements specified within the methodology and were modeled using the proposed modeling approach. Archetypes were designed for the Design Earthquake (DE) and were evaluated for Maximum Considered Earthquake (MCE). Static pushover analysis was performed to determine the period based ductility (μ_T) and over-strength factors (Ω). The Incremental Dynamic Analysis (IDA) was performed for a set of predefined earthquakes termed "Far-Field" earthquakes scaled to Maximum Considered Earthquake (MCE). The results of the IDA was used to plot the cumulative distribution function (CDF) which then leads to the determination of median collapse spectral acceleration ($\hat{S}_{CT}$) (Ibarra et al., 2002).

7.4.1 *Dynamic Analysis*

Incremental 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 due to the sites located 10 km or greater from the fault rupture. The selection criteria for these ground motion are discussed in Appendix A of the FEMA P695 report. The purpose of these ground motions is to provide consistent basis for collapse evaluation for structures in any seismic design category, in any seismic region and with any soil classification. The list of ground motions with the pertaining information are provided in Table 7.3.

Table 7.3. Far-field ground motions in FEMA P695

No.	Peer Rec. No.	Name	Recording Station	Year	M	PGA max*
1	953	Northridge	Beverly Hills – Mulhol	1994	6.7	0.52
2	960	Northridge	Canyon Country–WLC	1994	6.7	0.48
3	1602	Duzce, Turkey	Bolu	1999	7.1	0.82
4	1787	Hector Mine	Hector	1999	7.1	0.34
5	169	Imperial Valley	Delta	1979	6.5	0.35
6	174	Imperial Valley	El Centro Array#11	1979	6.5	0.38
7	1111	Kobe, Japan	Nishi – Akashi	1995	6.9	0.51
8	1116	Kobe, Japan	Shin – Osaka	1995	6.9	0.24
9	1158	Kocaeli, Turkey	Duzce	1999	7.5	0.36
10	1148	Kocaeli, Turkey	Arcelik	1999	7.5	0.22
11	900	Landers	Yermo Fire Station	1992	7.3	0.24
12	848	Landers	Coolwater	1992	7.3	0.42
13	752	Loma Prieta	Capitola	1989	6.9	0.53
14	767	Loma Prieta	Gilroy Array#3	1989	6.9	0.56
15	1633	Manjil, Iran	Abbar	1990	7.4	0.51
16	721	Superstition Hills	El Centro Imp. Co.	1987	6.5	0.36
17	725	Superstition Hills	Poe Road (temp)	1987	6.5	0.45
18	829	Cape Mendocino	Rio Dell Overpass	1992	7.0	0.55
19	1244	Chi-Chi, Taiwan	CHY 101	1999	7.6	0.44
20	1485	Chi-Chi, Taiwan	TCU045	1999	7.6	0.51
21	68	San Fernando	LA – Hollywood Stor.	1971	6.6	0.21
22	125	Friuli, Italy	Tolmezzo	1976	6.5	0.35

*PGA_{max} refers to larger value of the two recorded components

The engineering demand parameter (EDP) for IDA in this study is inter-story drift and the intensity measure (IM) was spectral acceleration. The ground motions were scaled to increasing intensities to determine median collapse intensity, $\hat{S}_{CT}$, which is defined as the intensity in which 22 of the 44 ground motions cause collapse. While a full IDA analysis is not necessary to determine median collapse intensity, $\hat{S}_{CT}$, for the purpose of this study ground motions were scaled to increasing intensities that ranged from 1g to 5 g. The IDA results were then used to develop collapse fragility that is defined as conditional probability of a limit state given a certain demand, for example in this case a specified inter-story drift limit is the limit state and spectral acceleration is the demand. A sample IDA with the corresponding fragility curve is shown in Figure 7.14.

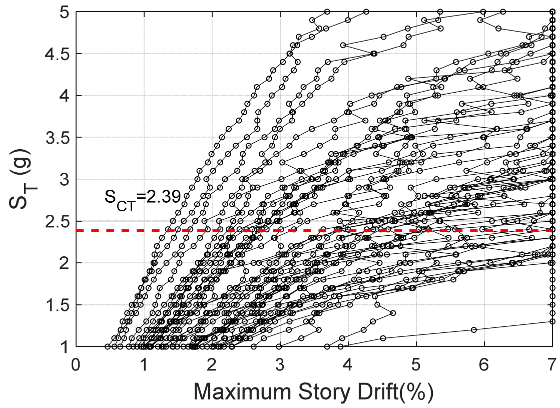
The scaling was performed in accordance with the FEMA P695 methodology where a record set is scaled by a single factor such that the median response spectrum of the normalized set matches the spectral acceleration of interest at the fundamental period of the building.

As mentioned before, the fundamental period T calculated based on the following. T is not permitted to exceed:

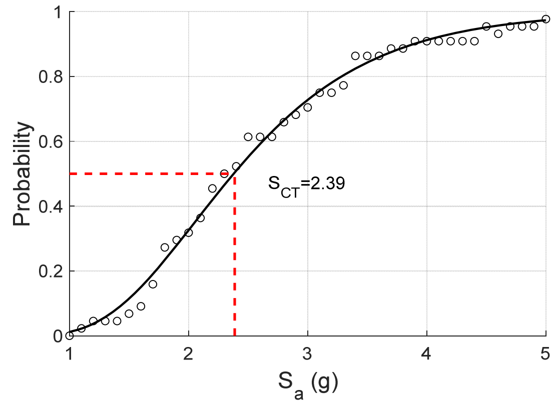
$$T = Cu\ Ta = Cu\ Ct\ h_n^x \geq 0.25\ sec$$

h_n = height of the structure, C_t=0.02 (ASCE Table 12.8-2), C_u=1.40 (ASCE Table 12.8-1), and x = 0.75 (ASCE Table 12.8-2). The upper limit is used since analytically T was found to be larger.

Figure 7.15 shows an example of these ground motions, scaled to MCE demand.



(a)



(b)

Figure 7.14. Sample results of nonlinear dynamic analysis (a) IDA (b) collapse fragility curve with lognormal fit

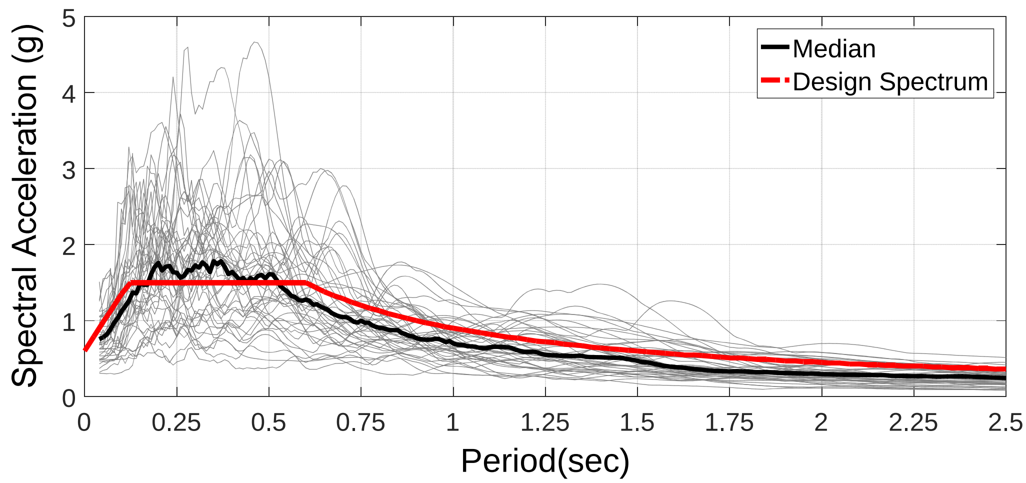


Figure 7.15. FEMA P695 far-field ground motions scaled to MCE demand at T=0.265 sec

7.4.2 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 the modeling purposes, hysteretic damping is directly considered in hysteretic modeling of the CLT shear walls while viscous damping is considered in the form of Rayleigh damping as a percentage of critical damping.

Based on a comprehensive study on light-frame wood structures (Camelo et al., 2002), the measured values of viscous damping ranged from 2.6% to 17.3% of the critical damping. The study included three distinct methods that were (a) measurements of the structural response under seismic event (b) forced vibration response (c) shake table response of a two-story structure. The seismic measurement performed for five buildings that ranged from 1 to 3 stories indicated damping ratios ranging between 6.3% to 17.3%. The damping ratios obtained from force vibration tests on two two-story and one three-story structure ranged from 2.6% to 6.8%. For the two-story structure the information on which is provided in Folz and Filiatrault (2004) the average damping ratio was close to 7%.

In the case of special CLT shear wall system, perpendicular walls, nonstructural components, and CLT walls in addition to the shear walls in a wall line contribute to the damping. According to the FEMA P695 methodology, the damping can be in the range of 2% to 5% of critical damping and for the purpose of this study the lower bound of 2% critical damping was assumed. It is important to point out that previous studies of SAPWood, cited earlier in Section 7.3, have shown appropriate consideration of the energy dissipation associated with various percent damping assumptions.

7.4.3 Static Pushover Analysis

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

Archetypes are subjected to vertically distributed load pattern proportional to their corresponding fundamental mode as the lateral displacement was increased monotonically. The numerical model for this purpose is developed in OpenSees (McKenna et al., 2011). The developed model is similar to the SAPWood model where diaphragms are assumed as rigid and

CLT shear walls are presented using the phenomenological CUREE-SAWS model. The model accounts for the degradation in stiffness and strength as well as the pinching behavior.

Second order (P-Δ) effects were considered in the pushover analysis using leaning columns (Geschwindner, 2000). 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.

Period based ductility is obtained from the pushover analyses using the following equation. A sample pushover curve is shown in Figure 7.16.

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{7.1}$$

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.

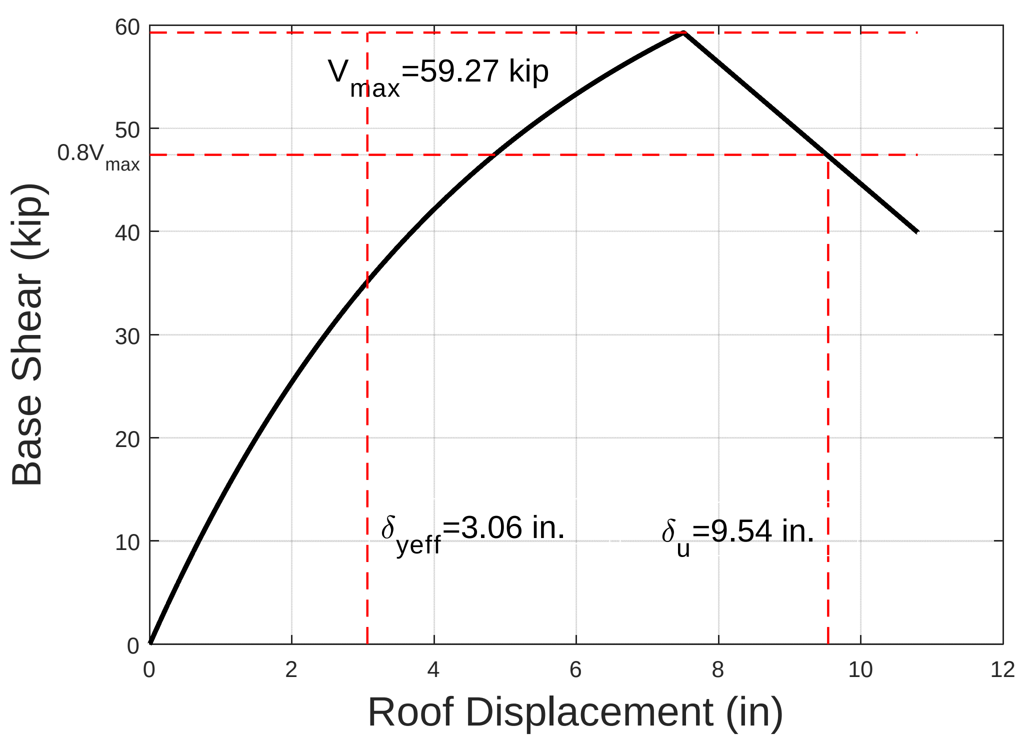


Figure 7.16 . Sample nonlinear static pushover analysis

CHAPTER 8: PERFORMANCE EVALUATION AND RESULTS

Collapse criteria were used with IDA results to develop collapse fragility which is expressed as the failure probability of the proposed seismic force resisting system with respect to the spectral acceleration. The seismic performance factors were evaluated and then 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 FEMA P695 methodology are repeated to obtain the desired ACMR.

8.1 Collapse Criteria

In order to evaluate collapse of CLT special shear wall system, failure criteria was defined in terms of inter-story drift with a limit was placed on it. Based on the CLT shear wall test results, shown in Table 8.1 and 8.2, an inter-story drift ratio of 4.5% and 5.5% were used for low aspect ratio and high aspect ratio cases, respectively.

Table 8.1. Maximum drift observed in tests

Test #	Length (ft)	No. connectors	Connector type	Δ_{max} (in.) +ve	Δ_{max} (in.) -ve	Δ_{max} (in.) Ave.	Drift (%)
03	4	3	A3	4.49	4.48	4.5	4.67
04	4	3	A3	4.5	4.49	4.5	4.68
05	4	3	A3	6	6	6.0	6.25
06*	4	3	A3	6	6	6.0	6.25
09	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 8.2. Drift data summary

Panel Aspect Ratio	Connector type	Ave. Drift (%)
4:1	A3	7.12
2:1		4.91

8.2 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 response of the index archetypes due to the 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 range of structural performance.

The total collapse uncertainty is given by the following equation:

$$\boldsymbol{\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}} \tag{8.1}$$

where β_{TOT} is the total system collapse uncertainty, β_{RTR} is the record-to-record collapse uncertainly, 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} is calculated for each archetype using the following equation and information on remaining uncertainty parameters are provide in Table 8.3-8.5.

$$\boldsymbol{\beta_{RTR} = 0.1 + \mu_T \leq 0.40} \tag{8.2}$$

Table 8.3. Quality rating of design requirements (After Table 3-1 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 8.4. Quality rating of test data (After Table 3-2 FEMA P695)

Completeness and robustness	Confidence in test results		
	High	Medium	Low
High: Material, component, connection, assembly and system behavior well understood and accounted for. All, or nearly all, important testing issues addressed.	Superior β_{TD}=0.10	Good β_{TD} =0.20	Fair β_{TD} =0.35
Medium: Material, component, connection, assembly and system behavior generally understood and accounted for. Most important testing issues addressed.	Good β_{TD} =0.20	Fair β_{TD} =0.35	Poor β_{TD} =0.50
Low: Material, component, connection, assembly and system behavior fairly understood and accounted for. Several important testing issues not addressed.	Fair β_{TD} =0.35	Poor β_{TD} =0.50	--

Table 8.5. Quality rating of index archetype models (After Table 5-3 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 β_{MDL}=0.10	Good β_{MDL} =0.20	Fair β_{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 β_{MDL} =0.20	Fair β_{MDL} =0.35	Poor β_{MDL} =0.50
Low: Significant aspects of the design space and/or collapse behavior are not captured in the index archetypes.	Fair β_{MDL} =0.35	Poor β_{MDL} =0.50	--

Collapse evaluation requires calculation of total uncertainty using Eq. 8.1 provided earlier. The quality ratings associated with the ground motion records, design requirements, test data, and modeling for the purpose of this study is shown in Table 8.6. The approach herein is similar to the light-frame wood structure example provide in FEMA P695. The quality ratings were selected considering the test data, design requirements that are based on historic design provisions of NDS, and the modeling approach.

Table 8.6. Quality ratings used for evaluation

Uncertainty	Quality rating value	Description
Record-to-record (β_{RTR})	0.40	Eq. 8.2
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

8.3 Collapse Margin Ratio and Adjusted Collapse Margin Ratio

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

$$CMR_i = \frac{\hat{S}_{CT\,i}}{S_{MT}} \tag{8.3}$$

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

CMR was then adjusted using Spectral Shape Factor, SSF, to account for the effects of spectral shape.

$$\boldsymbol{ACMR = CMR * SSF} \tag{8.4}$$

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 is obtained using the data provided in Table 8.7. As presented in Eq. 1.5 earlier in the report, code based period is calculated as follows:

$$T=Cu\ Ta=Cu\ Ct\ h_n^{x}=0.02*1.4*\ h_n^{0.75}=0.028\ h_n^{0.75}$$

where h_n is height of the structure

Table 8.7. Spectral Shape Factor (SSF) for Archetypes Designed using SDC D_{max} (After Table 7-1b FEMA P695)

T (sec)	Period based ductility, μ_T							
	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

8.4 Overstrength Factor, Ω_o

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

$$\Omega = \frac{V_{max}}{V} \quad \textbf{(8.5)}$$

where V_{max} represents the maximum strength of the full-yielded system that is obtained from static pushover analysis described earlier in Section 7.4.3 and V is the seismic base shear required for design.

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

8.5 Deflection Amplification Factor, C_d

The deflection amplification factor, C_d, is calculated using the equation shown below.

$$C_d = \frac{R}{B_I} \tag{8.6}$$

As indicated in the FEMA P695 document, inherent damping, β_I, maybe assumed to be 5% of critical damping which gives a corresponding value of B_I=1.0 based on Table 18.7-1 of ASCE 7-16. This results in C_d=R.

The example application presented in Appendix F 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.

8.6 Results

8.6.1 Response Modification Factor, R, for CLT Special Shear Walls

The procedure presented in the previous chapter was carried out for R=3 for all the archetypes and for R=4 only for archetypes with high aspect ratio panel configurations. Seismic design properties, final results and acceptance criteria for are provided in Tables 8.8-8.11 and Tables 8.12-8.16 for R=3 case and in Tables 8.17-8-20, respectively. Pushover curves and collapse fragility curves for all the archetypes are provided in Appendix G.

Based on the results shown in Table 8.12-8.16, all the individual archetypes designed for R=3 pass the $ACMR_{20\%}$ criteria and average of the performance groups exceed the $ACMR_{10\%}$

criteria for the performance groups. Similarly, looking at the results shown in Tables 8.19-8.20 for R=4 for archetypes with high aspect ratio panel configurations, all the individual archetypes and performance groups pass their corresponding criteria. The summary of the ACMR for the performance groups are provided in Figure 8.1 and 8.2 for the R=3 and R=4 cases, respectively.

8.6.2 Overstrength Factor, Ω_o, CLT Special Shear Walls

For R=3 case for all the archetypes, the overstrength factor calculated for the archetypes range from 1.8 to 4.85 with most values centered around 3 and the average overstrength factors for the performance groups range from 2.29 to 3.53. Similarly for R=4 case for archetypes with high aspect ratio panel configurations, the overstrength factor 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. Since the largest average of the performance groups is greater than 3, the upper limit of $\Omega_o=3.0$ is considered for the system for both cases.

8.6.3 Deflection Amplification Factor for CLT Special Shear Walls

As explained in Section 8.5 of the document, with B_I=1.0, $C_d=R$. Therefore, for the R=3 case, C_d=3.0. It is important to note that in the case of R=4, archetypes were designed using C_d=3 but that C_d=4 is proposed for purposes of design and that the effect of C_d=4 would be to make designs stronger in order to meet seismic drift limits.

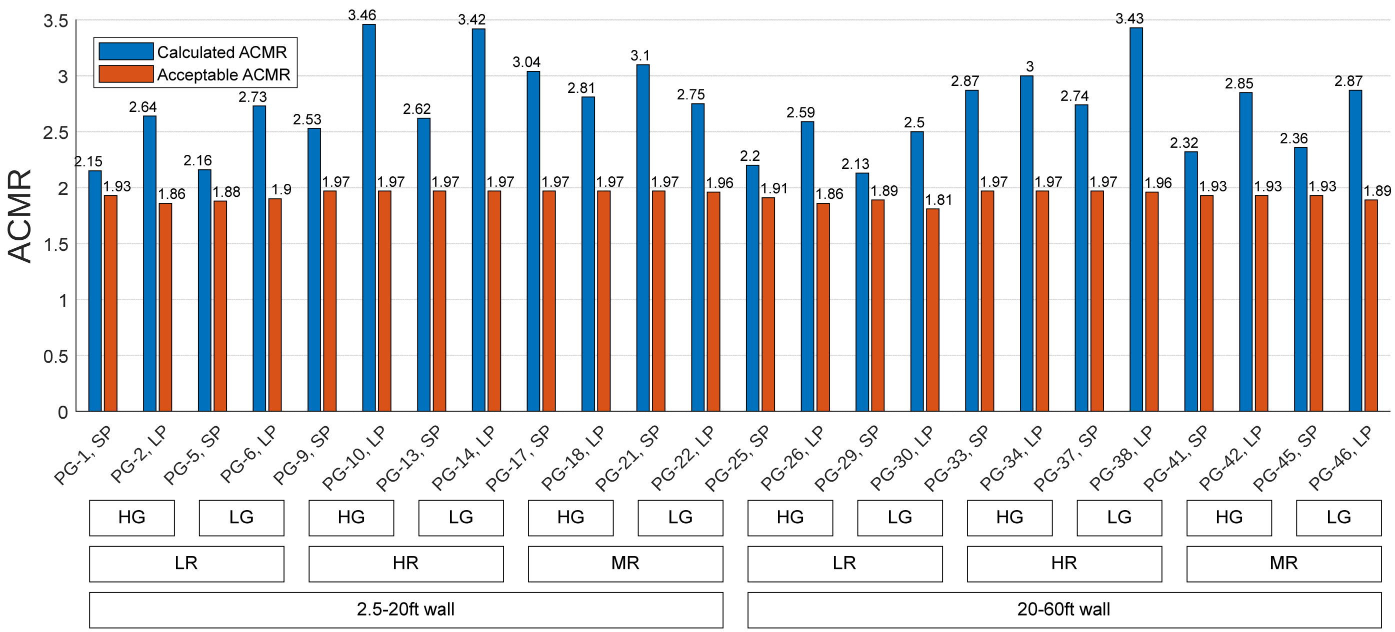


Figure 8.1. Summary of ACMR for performance groups

R=3, ξ=0.02, Φ=0.50, NSC (Non-simulated collapse) =4.5% Inter-story drift for LR and 5.5% for HR

HG (High Gravity), LG (Low Gravity), LR (Low aspect Ratio), HR (High aspect Ratio), MR (Mixed aspect Ratio)

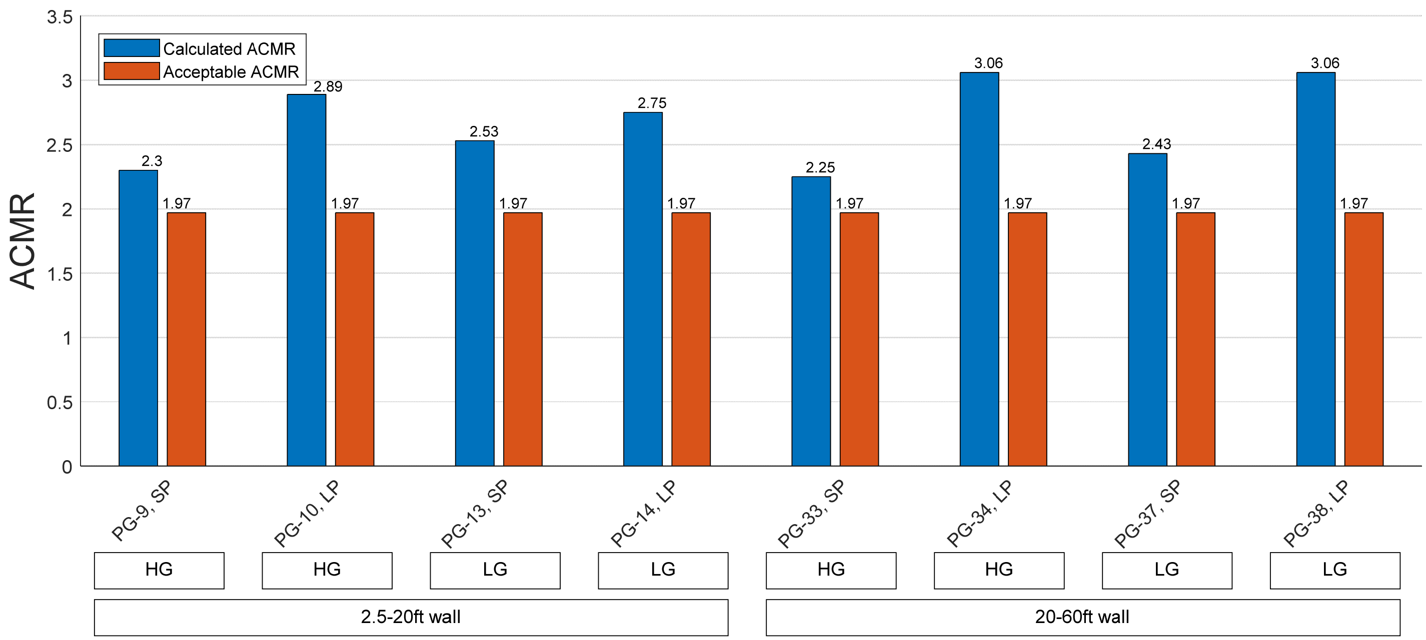


Figure 8.2. Summary of ACMR for HR (High aspect Ratio) panel performance groups

R=4, ξ=0.02, Φ=0.50, NSC (Non-simulated collapse) =5.5% Inter-story drift for HR

HG (High Gravity), LG (Low Gravity)

Table 8.8. Archetype Seismic Design Properties, R=3

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
PG-01									
1	1_E_3_1_LR_HG_DX_SP	Low	High	D_{max}	0.36	0.59	17.43	53.9	1.5
2	2_4_2_1_LR_HG_DX_SP	Low	High	D_{max}	0.26	0.53	19.30	67.2	1.5
3	6_D_2_1_ LR_HG_DX_SP	Low	High	D_{max}	0.26	0.47	8.20	24.6	1.5
PG-02									
4	4_3_ 6_1_LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.77	34.79	105	1.49
5	8_3_6_1_ LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.54	30.31	90.5	1.49
6	9_B_6 _1_ LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.60	34.64	104.6	1.49
PG-05									
13	1_E_3_1_LR_LG_DX_SP	Low	Low	D_{max}	0.36	0.61	14.83	46.0	1.50
14	2_4_2_1_LR_LG_DX_SP	Low	Low	D_{max}	0.26	0.53	17.33	61.34	1.50
15	6_D_2_1_ LR_LG_DX_SP	Low	Low	D_{max}	0.26	0.49	7.14	21.4	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}	0.36	0.53	17.43	53.9	1.50
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	0.26	0.47	19.30	67.2	1.50
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	0.26	0.48	8.20	24.6	1.50
PG-10									
28	4_3_ 6_1_HR_HG_ DX_LP	High	High	D_{max}	0.604	0.74	34.79	105	1.49
29	8_3_6_1_ HR_HG_ DX_LP	High	High	D_{max}	0.604	0.66	30.31	90.5	1.49
30	9_B_6 _1_HR_HG_ DX_LP	High	High	D_{max}	0.604	0.67	34.64	104.6	1.49

Table 8.9. Archetype Seismic Design Properties, R=3

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
			PG-13						
37	1_E_3_1_HR_LG_DX_SP	High	Low	D_{max}	0.36	0.55	14.83	46.02	1.50
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.26	0.46	17.33	61.38	1.50
39	6_D_2_1_ HR_LG_DX_SP	High	Low	D_{max}	0.26	0.47	7.10	21.4	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	115.8	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	161.9	1.50
			PG-18						
52	4_3_ 6_1_MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.72	34.79	105	1.49
53	8_3_6_1_ MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.58	30.31	90.5	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}	0.45	0.64	32.17	96.52	1.50
62	4_3_ 4_1_MR_LG_ DX_SP	Mix	Low	D_{max}	0.45	0.62	37.71	113.12	1.50
63	7_3_3_1_MR_LG_DX_SP	Mix	Low	D_{max}	0.36	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 8.10. Archetype Seismic Design Properties, R=3

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
			PG-25						
73	2_2_2_2_ LR_HG_ DX_SP	Low	High	D_{max}	0.26	0.48	24.16	71.7	1.50
74	3_2_1_2_ LR_HG_ DX_SP	Low	High	D_{max}	0.25	0.29	14.73	44.2	1.50
75	5_B_2_2_ LR_HG_ DX_SP	Low	High	D_{max}	0.26	0.47	62.73	188.2	1.50
			PG-26						
76	4_E_6_2_ LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.77	54.77	165.3	1.49
77	8_B_6_2_ LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.55	61.70	184.3	1.49
78	9_3_6_2_ LR_HG_ DX_LP	Low	High	D_{max}	0.604	0.50	98.56	297.6	1.49
			PG-29						
85	2_2_2_2_ LR_LG_ DX_SP	Low	Low	D_{max}	0.26	0.46	21.71	64.32	1.50
86*	3_2_1_2_ LR_LG_ DX_SP	Low	Low	D_{max}	0.25	0.29	14.73	44.2	1.50
87	5_B_2_2_ LR_LG_ DX_SP	Low	Low	D_{max}	0.26	0.51	53.78	161.4	1.50
			PG-30						
88	4_E_6_2_ LR_LG_ DX_LP	Low	Low	D_{max}	0.604	0.77	45.51	137.4	1.49
89	8_B_6_2_ LR_LG_ DX_LP	Low	Low	D_{max}	0.604	0.54	50.22	151.6	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}	0.36	0.52	26.02	74.4	1.50
98	6_E_2_2_ HR_HG_ DX_SP	High	High	D_{max}	0.26	0.45	23.15	69.6	1.50
99	7_A_3_2_ HR_HG_ DX_SP	High	High	D_{max}	0.36	0.53	79.24	237.7	1.50
			PG-34						
100	4_E_6_2_ HR_HG_ DX_LP	High	High	D_{max}	0.604	0.69	54.77	165.3	1.49
101	8_B_6_2_ HR_HG_ DX_LP	High	High	D_{max}	0.604	0.64	61.70	184.3	1.49
102	9_3_6_2_ HR_HG_ DX_LP	High	High	D_{max}	0.604	0.63	98.56	297.6	1.49

*Archetype 86 is same as Archetype 74 since for the one-story archetype LG and HG cases are the same

Table 8.11. Archetype Seismic Design Properties, R=3

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
			PG-37						
109	1_A_3_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.36	0.49	22.15	63.36	1.50
110	6_E_2_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.26	0.50	20.19	60.57	1.50
111	7_A_3_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.36	0.52	66.82	200.4	1.50
			PG-38						
112	4_E_6_2_ HR_LG_ DX_LP	High	Low	D_{max}	0.604	0.75	45.52	137.4	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}	0.26	0.46	24.16	71.7	1.50
122*	3_2_1_2_ MR_HG_ DX_SP	Mix	High	D_{max}	0.25	0.28	14.73	44.2	1.50
123	5_B_2_2_ MR_HG_ DX_SP	Mix	High	D_{max}	0.26	0.50	62.73	188.2	1.50
			PG-42						
124	4_E_6_2_ MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.74	54.77	165.3	1.49
125	8_B_6_2_ MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.60	61.70	184.3	1.49
126	9_3_6_2_ MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.56	98.56	297.6	1.49
			PG-45						
133	2_2_2_2_ MR_LG_ DX_SP	Mix	Low	D_{max}	0.26	0.46	21.71	64.32	1.50
134	3_2_1_2_ MR_LG_ DX_SP	Mix	Low	D_{max}	0.25	0.28	14.73	44.2	1.50
135	5_B_2_2_ MR_LG_ DX_SP	Mix	Low	D_{max}	0.26	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	137.4	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

*Archetype 134 is same as Archetype 122 since for the one-story archetype LG and HG cases are the same

Table 8.12. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=3

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/Fail
					PG-01							
01	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
02	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
03	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 Performance Group:				**3.03**			**1.84**		**2.15**	**1.93**	**PASS**
					PG-02							
04	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
05	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
06	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 Performance Group:				**3.11**			**2.27**		**2.64**	**1.864**	**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 Performance Group:				**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 Performance Group:				**3.27**			**2.32**		**2.73**	**1.9**	**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 Performance Group:				**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 Performance Group:				**2.90**			**2.76**		**3.46**	**1.97**	**PASS**

Table 8.13. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=3

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**2.77**			**2.50**		**3.10**	**1.97**	**PASS**

Table 8.14. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=3

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**2.73**			**2.18**		**2.50**	**1.81**	**PASS**

Table 8.15. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=3

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**3.14**			**1.93**		**2.32**	**1.93**	**PASS**

Table 8.16. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=3

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	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 Performance Group:					**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 Performance Group:					**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 Performance Group:					**3.00**			**2.43**		**2.87**	**1.89**	**PASS**

Table 8.17. Archetype Seismic Design Properties, R=4

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
				PG-09					
25	1_E_3_1_HR_HG_DX_SP	High	High	D_{max}	0.36	0.62	13.05	53.90	1.50
26	2_4_2_1_HR_HG_DX_SP	High	High	D_{max}	0.26	0.56	14.48	67.20	1.50
27	6_D_2_1_HR_HG_DX_SP	High	High	D_{max}	0.26	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}	0.36	0.63	11.12	46.02	1.50
38	2_4_2_1_HR_LG_DX_SP	High	Low	D_{max}	0.26	0.49	13.00	61.38	1.50
39	6_D_2_1_ HR_LG_DX_SP	High	Low	D_{max}	0.26	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}	0.36	0.64	19.49	74.40	1.50
98	6_E_2_2_ HR_HG_ DX_SP	High	High	D_{max}	0.26	0.57	17.39	69.60	1.50
99	7_A_3_2_ HR_HG_ DX_SP	High	High	D_{max}	0.36	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 8.18. Archetype Seismic Design Properties, R=4

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
				PG-37					
109	1_A_3_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.36	0.60	16.61	63.36	1.50
110	6_E_2_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.26	0.61	15.14	60.57	1.50
111	7_A_3_2_ HR_LG_ DX_SP	High	Low	D_{max}	0.36	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

Table 8.19. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=4

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/Fail
					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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**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 Performance Group:				**3.61**			**2.53**		**3.06**	**1.97**	**PASS**

Table 8.20. Summary of Static and Dynamic Analysis, Collapse Margins, and Acceptance Criteria, R=4

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	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 Performance Group:				**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 Performance Group:				**3.41**			**2.53**		**3.06**	**1.97**	**PASS**

CHAPTER 9: SUMMARY, CONCLUSIONS, RECOMMENDATIONS AND CLT OUTLOOK

Seismic performance factors are currently not available for CLT shear wall systems in US building codes and its reference design standards. The purpose of this study was to investigate the seismic behavior of CLT shear walls and determine seismic design factors namely, the response modification factor (R-factor), the system overstrength factor (Ω), and the deflection amplification factor (C_d) for CLT shear walls in platform type construction using the FEMA P695 process. Results from the study will be proposed for implementation in US building codes (such as the International Building Code, IBC) and its 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 (SDPWS). The approach adopted in this study is summarized and conclusions are provided as follows:

9.1 Summary

- Index archetypes that include single family dwellings, multi-family dwellings, and commercial buildings were developed to comprehensively represent the anticipated design space for the proposed seismic force resisting system. In total 9 index buildings and 72 archetypes were used in this study. The archetypes were then systematically extracted from the index buildings and categorized into various performance groups that reflect common behavioral characteristics, consistent with the FEMA P695 methodology.
- Tests were performed on the angle brackets and inter-panel connectors prescribed as part of the CLT shear wall design method. These two types of generic connectors were manufactured from sheet steel in the machine shop at CSU and commodity nails were used in the metal connectors. Shear and uplift tests were performed on angle brackets and

shear tests were performed on the inter-panel connectors. Connectors were designed based on NDS to ensure nail yielding which is a desired fastener yield limit state due to the ductility associated with it.

- Quasi-static cyclic tests were conducted on a suite of CLT shear walls designed in accordance with the CLT shear wall design method to systematically investigate the effects of various parameters that include: boundary condition of the CLT shear wall imposed by the CLT diaphragm, presence of gravity loading, connector type, connector plate thickness, CLT grade, CLT panel aspect ratio, CLT panel thickness, and presence of inter-panel connector (vertical joint). CLT shear wall tests were performed with the same generic connectors used in the connector testing.
- Prior to this study, little information was available related to the failure modes and collapse mechanisms for CLT shear walls, and even less about collapse of CLT as a structural system. Shake table tests were performed with three different configurations in accordance with the CLT shear wall design method that included approximately 4:1 aspect ratio panels (height/width=h/b), 2:1 aspect ratio panels, and 4:1 aspect ratio panels with perpendicular walls. The structure was subjected to ground motions scaled to intensities corresponding to a service level earthquake, design earthquake, and maximum considered earthquake, which in turn corresponds to a mean return period of 72 years, 474 years, and 2475 years, respectively.
- The design method for CLT shear walls used in this study was developed to be in accordance with the current applicable codes and standards and augmented with provisions to ensure the prescribed nominal unit shear capacity of the CLT shear wall is developed. Archetypes were designed in accordance with the proposed design

methodology and nonlinear numerical models were developed based on those archetype designs. The phenomenological CUREE-SAWS model was used to characterize CLT shear wall hysteretic behavior.

- Nonlinear analysis performed on the archetype models consisted of static pushover and dynamic (nonlinear time history) analysis. The former was used to determine period based ductility (μ_T) and over-strength factors (Ω) and the latter, performed in context of Incremental Dynamic Analysis (IDA) for the far-field ground motion set, was used to determine the median value of the collapse spectral acceleration ($\hat{S}_{CT}$) which was then used in calculating the collapse margin ratio (CMR) and adjusted collapse margin ratio (ACMR).

9.2 Conclusions

This study aimed to provide a systematic understanding of a new design method for CLT shear walls including the development of the seismic performance factors with the specific results highlighted as follows:

- The index archetypes developed as part of this study demonstrated acceptable collapse performance of the CLT shear wall system concept and range of applications including use in single family dwelling, multi-family dwelling, and commercial building index models.
- Connector thickness is important in obtaining the desired ductile behavior. The A1 connector with the thickness of 3/8 in. did not perform well and nail shear was observed. However, the A3 and B3 connectors made of 12 gauge steel (0.105 in.) demonstrated the desired behavior where the nonlinear behavior was due to the nail bending, wood bearing deformation around the nail, nail withdrawal, and metal connector deformation. Generic

connectors addressed by the design methodology utilize commodity materials whose design properties are addressed by building code referenced standards. The tested performance of the generic connectors establishes a performance baseline for proprietary alternatives to demonstrate equivalence.

- Isolated CLT shear wall testing demonstrated the following:
  - Similar to the finding of other tests cited in this dissertation, CLT demonstrated rigid behavior and energy dissipation occurred through the connectors with the nail yielding as the mechanism.
  - A special boundary condition that aimed to replicate diaphragm stiffness and restrict panel rotation by imposing vertical load and stiff loading beam, was shown to have a slightly beneficial effect on the CLT wall behavior.
  - Both stiffness and strength increased with the increase in gravity load; however, the change in the latter was observed to be less significant.
  - When using the exact same connectors and fasteners CLT grade was found to have an influence on strength and stiffness, which was attributed to specific gravity of the CLT grade used.
  - Panel thickness had only a slight effect on wall stiffness and strength.
  - High aspect ratio panels (4:1) had significantly less stiffness but had more deformation capacity than the low aspect ratio panels (2:1).
  - Walls comprised of high aspect ratio panels that are connected through vertical joints exhibited larger deformation capacity than low aspect ratio panels (2:1).

    - High aspect ratio panels (4:1) in a multi-panel shear wall with panels connected through vertical joints exhibited unit shear strength and stiffness proportional to shear wall length.
- The design method developed for CLT special shear wall systems was validated using the test results. With the exception of one test (1:1 aspect ratio wall), only high (4:1) and moderate (2:1) aspect ratio walls were investigated as part of this study. It was observed that longer walls (1:1) exhibited more sliding and less rocking behavior than other higher aspect ratio shear walls. The intent of this study and test program as a whole was to promote rocking behavior in the CLT shear wall system by controlling CLT panel aspect ratio and making longer wall assemblies by connecting high aspect ratio CLT panels at vertical joints between panels.
- System level demonstration of the CLT shear wall in platform construction was performed via shake table testing. CLT shake table tests conducted as part of this study were all successful and demonstrated that CLT shear wall system designed in accordance with the design methodology can meet the life-safety code requirements and can effectively be used in US seismic regions. The shake table specimens were designed with an R=4 and ϕ=0.55 which is significantly less conservative than the final proposed values outlined below. Transverse walls were shown to improve performance of the structure and give an insight into improved 3D vs 2D behavior of CLT systems.
- The seismic performance factors for the proposed CLT shear wall seismic force resisting system designed in accordance with the prescribed design methodology are recommended as follows:

Response modification factor (R-factor) = 3 for all configurations and R=4 only for cases with high aspect ratio panel configurations

System overstrength factor (Ω) =3

Deflection amplification factor (C_d) =3 for R=3 case and C_d=4 for R=4 case

These seismic performance factors will be proposed for recognition in US building codes enabling engineers to utilize CLT for seismic force resistance as part of the vertical seismic force resisting system without the need for special alternative methods and materials approvals for each design. It will also help in promoting use of this innovative and sustainable construction product.

9.3 Recommendations for future research

As mentioned earlier in the document, CLT is a fairly new product in the global construction market and even more so in the US. CLT related research in general and more specifically seismic related research will be of significant importance in further advancement of this innovative product. The research topics presented below range from small to large scale, i.e. component to assembly level, and are deemed important for further studies.

- Isolated CLT shear wall tests showed a possible lower bound of 2:1 panel aspect ratio for sliding dominant behavior. Only one test with 1:1 aspect ratio was evaluated in this study. Additional testing of 1:1 aspect ratio CLT shear walls will provide additional verification that the lower bound of 2:1 is appropriate and will provide additional information to potentially extend the design methodology to 1:1 aspect ratio panels.
- Certain assumptions were made regarding compression perpendicular to the grain of CLT for overturning moment calculations. Some referenced studies showed large variability in compression perpendicular to the grain due to its dependence on geometry, loading area

and orientation. Crushing deformations and ultimate strength limit states of the wall/floor panel will need to further studied in context of different in situ conditions to address likely conservatisms in current design.

- Seismic detailing is extremely important for proper implementation of new CLT systems and developing these details to ensure desired ductile behavior will require close collaboration between many researcher and practitioners
- The shake table tests with transverse walls provided insight into three-dimensional behavior of a CLT system. Further tests should be explored to investigate the beneficial effects of transverse walls (oriented perpendicular to and receiving out-of-plane loading) on CLT shear wall system response.
- For the purpose of this study, an inter-story drift limit based on the CLT shear wall tests conducted as part of this study was used to establish the non-simulated collapse limit state. Shear connection yielding and failure (such as nail withdrawal and nail bending and nail shear were observed), collapse mechanisms were not observed during either CLT wall tests or shake table tests. Identification of specific performance objectives, such as to limit damage associated with drift, will prove beneficial in CLT system design and could potentially serve as a starting point for performance based design (PBD) of CLT structures.
- As mentioned earlier in this dissertation, in its current form this study and the design methodology are intended for platform-type construction only. Further extension of this study along with the topic mentioned above will be of great importance and interest from engineering standpoint.

- Although there has been some testing done on CLT diaphragms, a more systematic study to investigate the interaction of CLT shear walls and CLT diaphragms, perform large and small scale testing, and develop and validate design models for CLT diaphragms is of interest.

9.4 CLT Outlook

CLT related developments outside of Europe, e.g. North America, Japan, New Zealand, and Australia, and a double digit annual growth rate has led to expectation that CLT may become as mainstream as glulam (Brandner et al., 2016). Global CLT production in 2015 was estimated at 650,000-700,000 m^3 and is projected to reach about 1.25 million m^3 by2020 in Europe only (UNFAO, 2017). The outlook for CLT is positive considering the current urbanization trend (Alig et al., 2004) and recovery of the US construction market from the recession of 2009-2012 with multi-family permits already fully recovered (Grasser, 2015). Some preliminary studies have shown that even in the current US construction market, the cost of CLT can be comparable to that of the reinforced concrete option (Sellen Construction, 2013; Mahlum Architects et al., 2014). This will further improve as new local manufacturers emerge and the CLT market grows. As of September 2018, there are four CLT manufacturers in North America that include Nordic, Structurlam, SmartLam, and D.R. Johnson and are all recognized to produce CLT in accordance with ANSI/APA PRG 320.

Mallo and Espinoza (2015) conducted a research to evaluate market potential and barriers to the widespread adoption of CLT. This was done through a nationwide survey to determine the level of awareness, perception and willingness to adopt CLT. Perception and willingness to use CLT on projects were found to have a significant relationship with the level of awareness meaning that CLT received favorable results in terms of perceived structural performance and likelihood of adoption from respondents who were more familiar with CLT.

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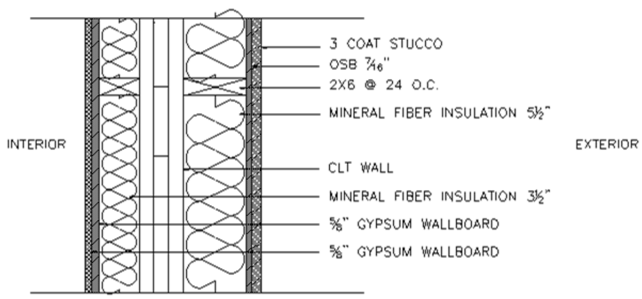
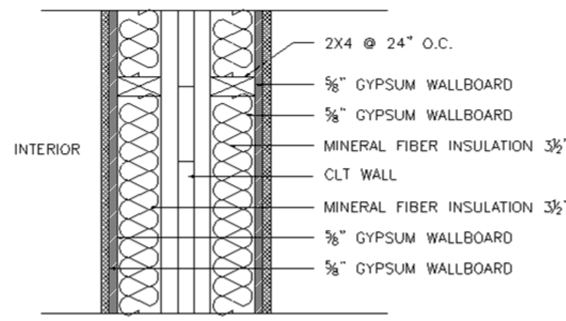
APPENDIX A: GRAVITY LOAD AND SEISMIC WEIGHT

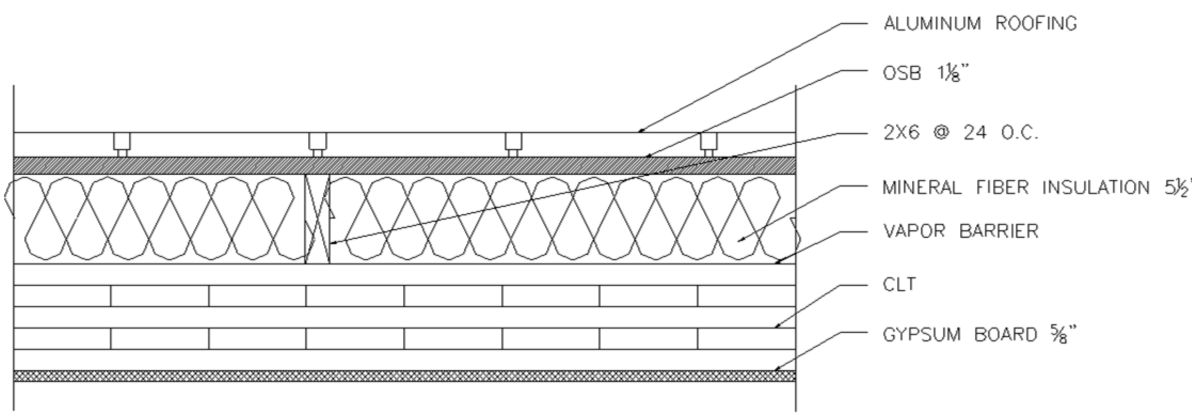
Figure A.1 and Table A.1 present typical composition of the exterior wall, interior wall, floor and roof materials and the corresponding unit weights that are used to calculate the seismic weight of the structure. It should be noted that these assigned layers are for the purpose of this study only and are not verified for fire and sound performance.



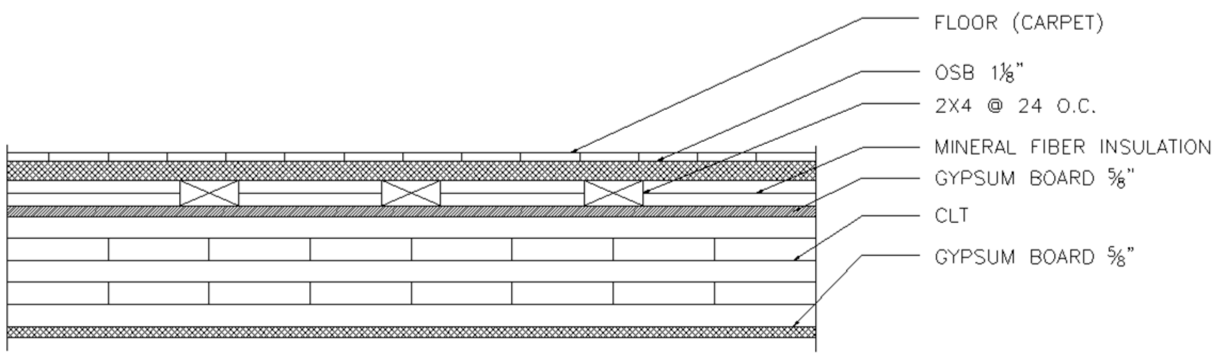
(a) Exterior wall



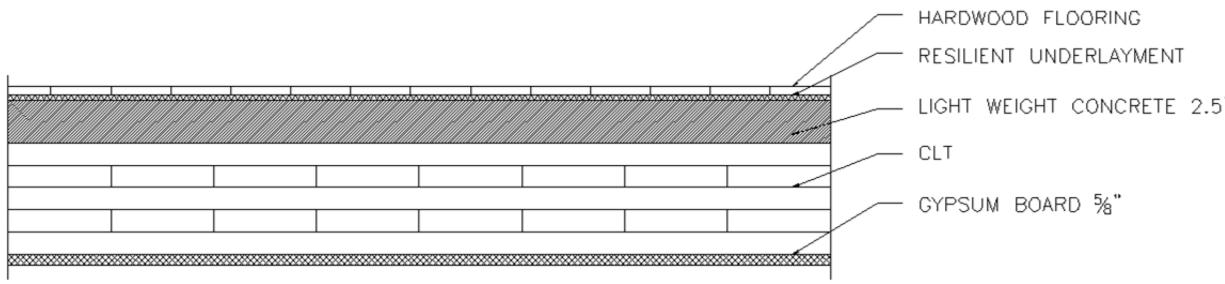
(b) Interior wall



(c) Roof



(d) Floor (Light)



(e) Floor (Heavy)

Figure A. 1. Wall and floor typical composition for weight calculations

Table A. 1. Dead load for structural and non-structural components

Material	Weight(psf)	Ref
Exterior wall		
Gypsum wallboard 5/8 in.	2.60	Table C3-2 ASCE 7
Gypsum wallboard 5/8 in.	2.60	"
2x4 @ 24 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
2x6 @ 24 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. (3 layers of 1.375)	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	"
2x4 @ 24 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
2x4 @ 24 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. (3 layers of 1.375)	9.625	NDS Supplement equation for density

Floor (Light)		
Floor (carpet)	2	
OSB 1 1/8 in.	3.375	Table C3-1 ASCE 7
2x4 @ 24 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
2x6 @ 24 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: COMPLETE SET OF INDEX BUILDINGS, ARCHETYPE DRAWINGS, AND LOAD CALCULATIONS

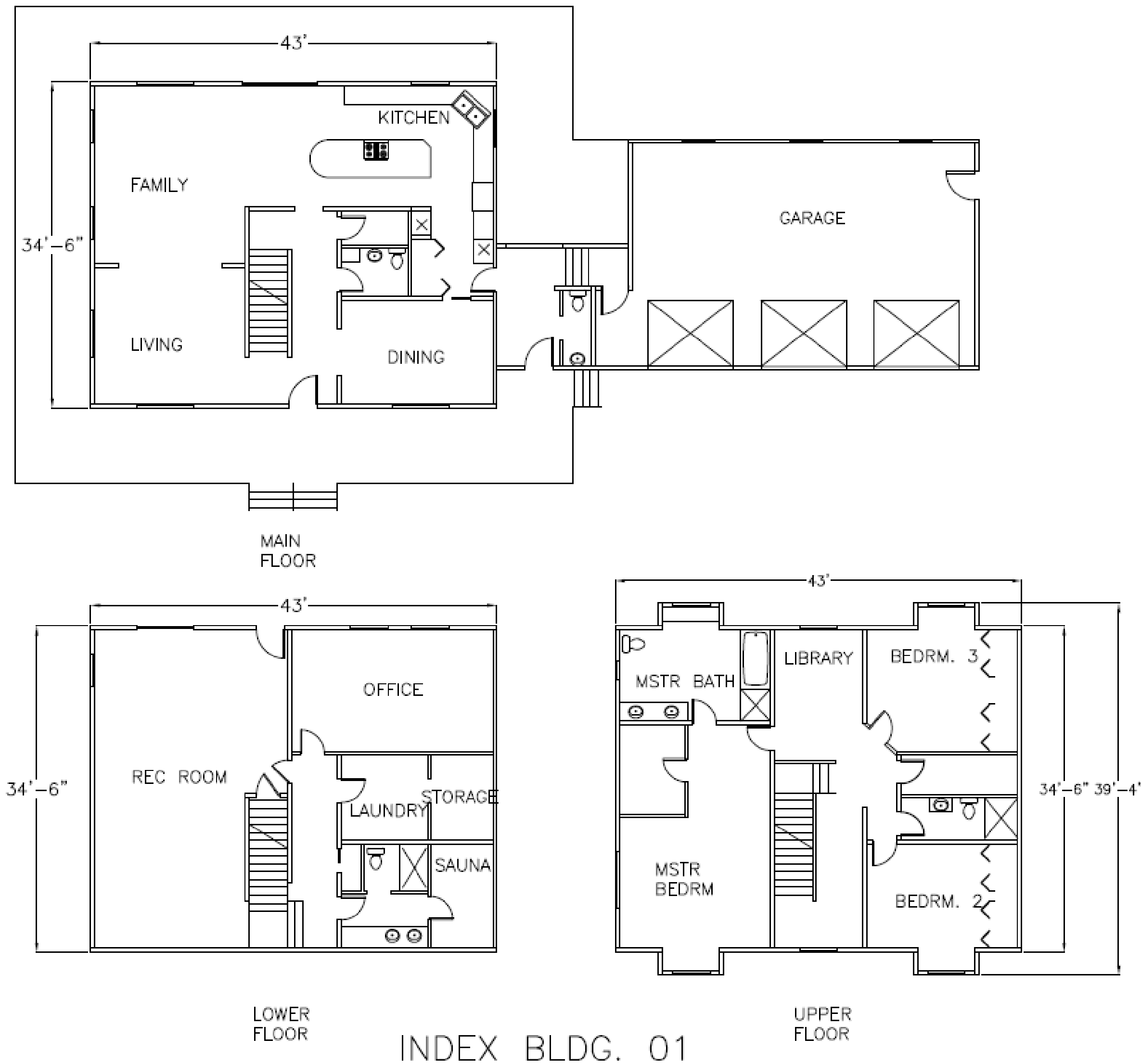


Figure B. 1. Index Bldg. 1 floor plan

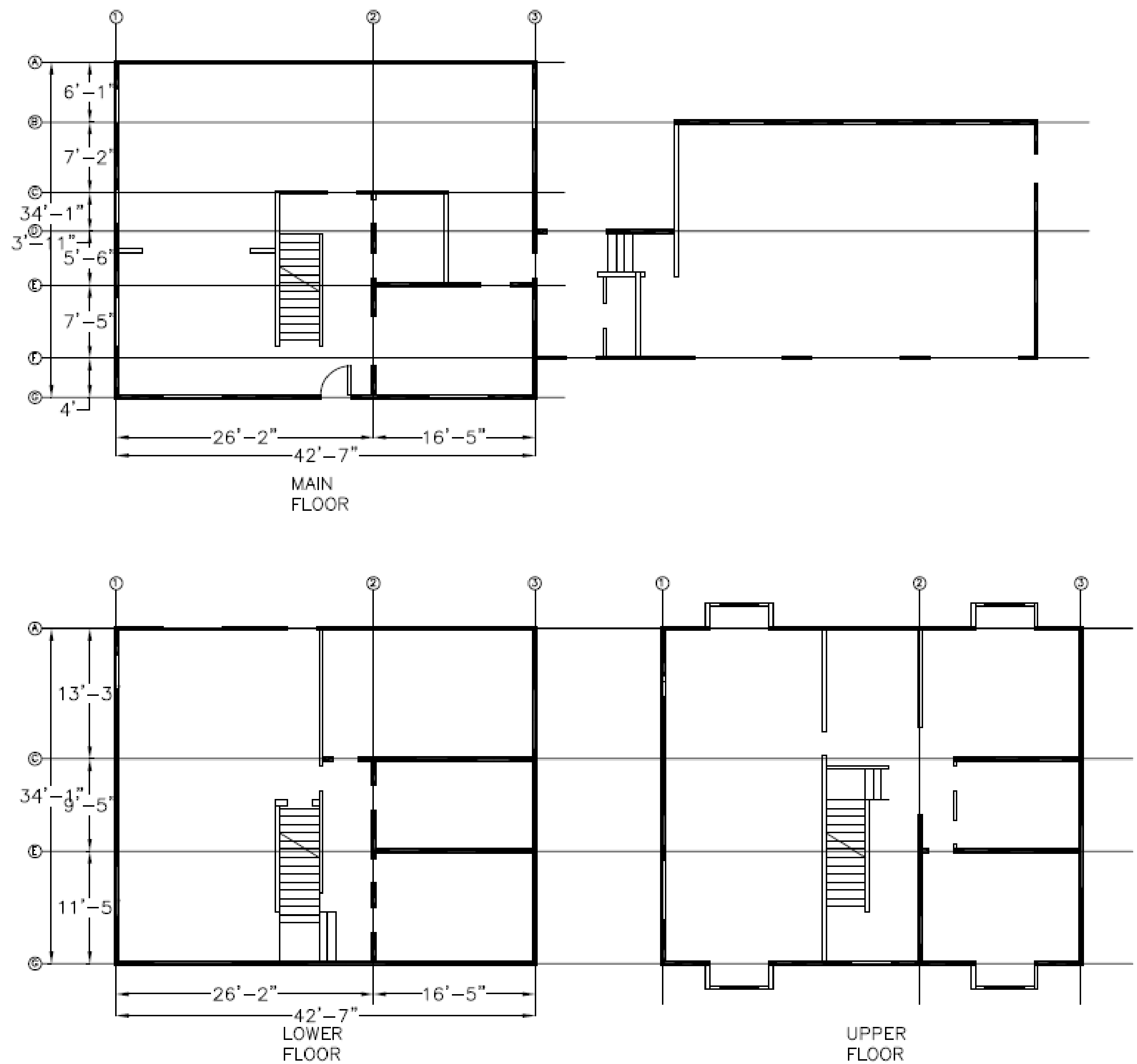


Figure B. 2. Index Bldg. 1 assigned shear walls

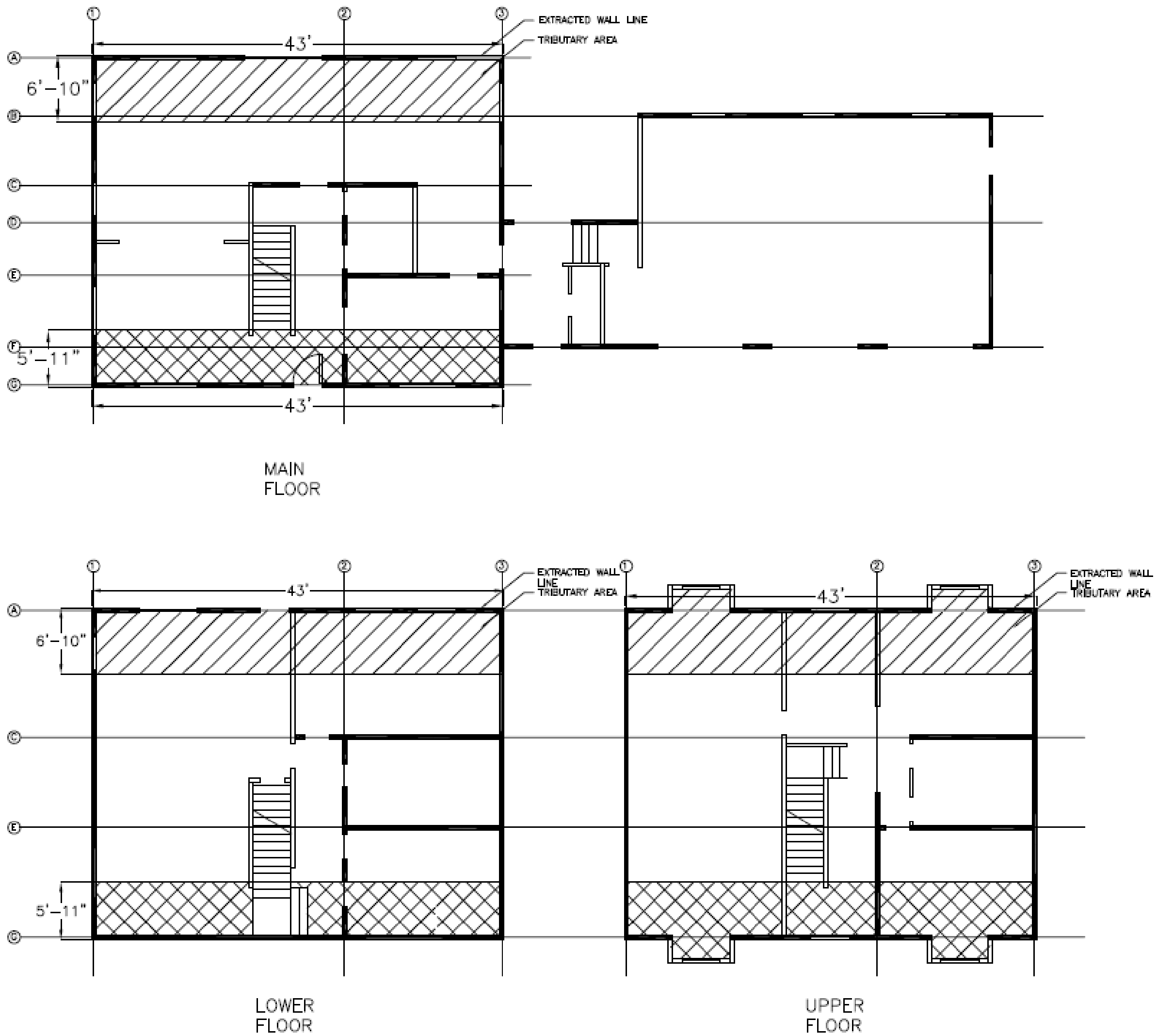


Figure B. 3. Index Bldg. 1 tributary area

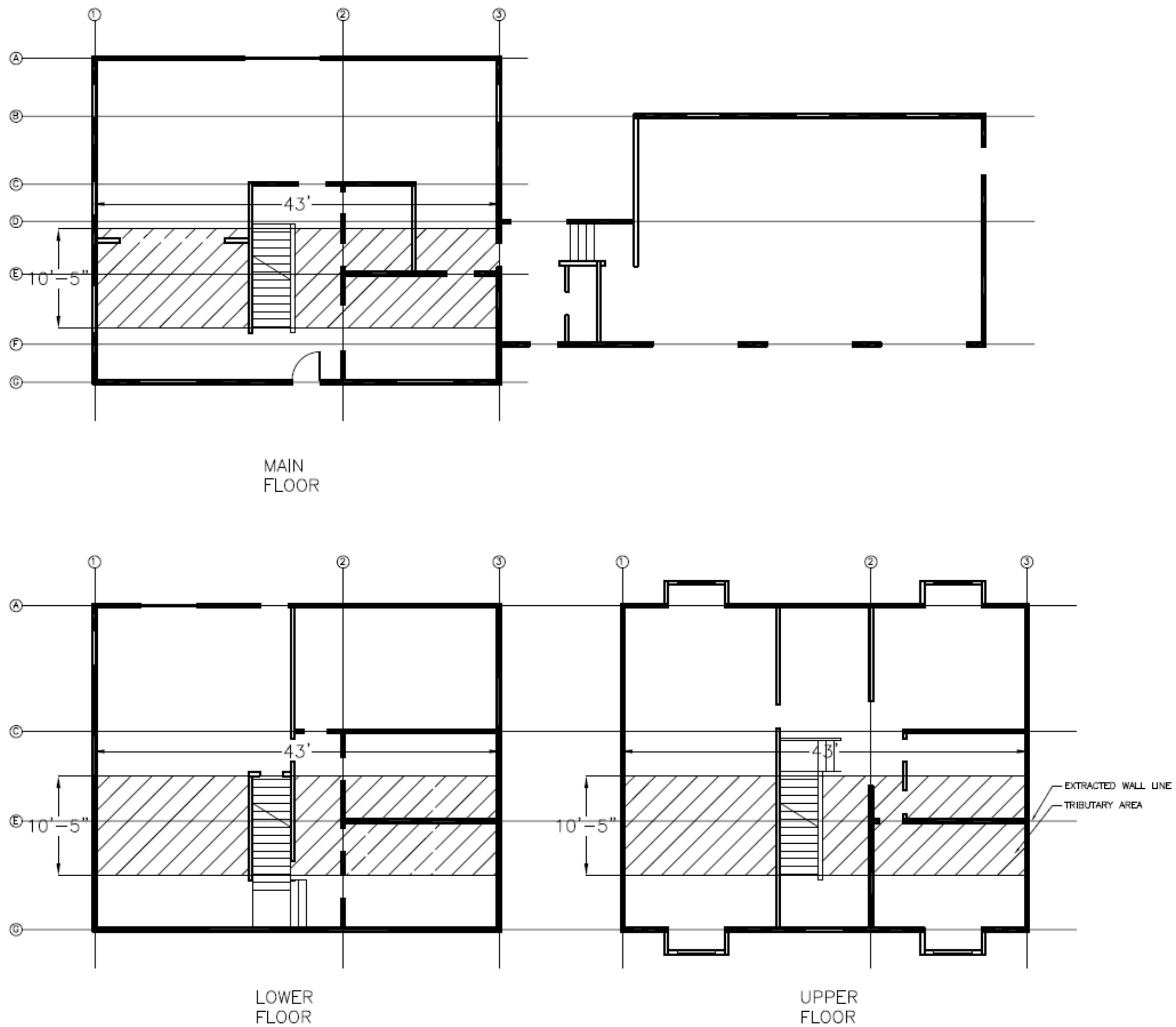


Figure B. 4. Index Bldg. 1 tributary area

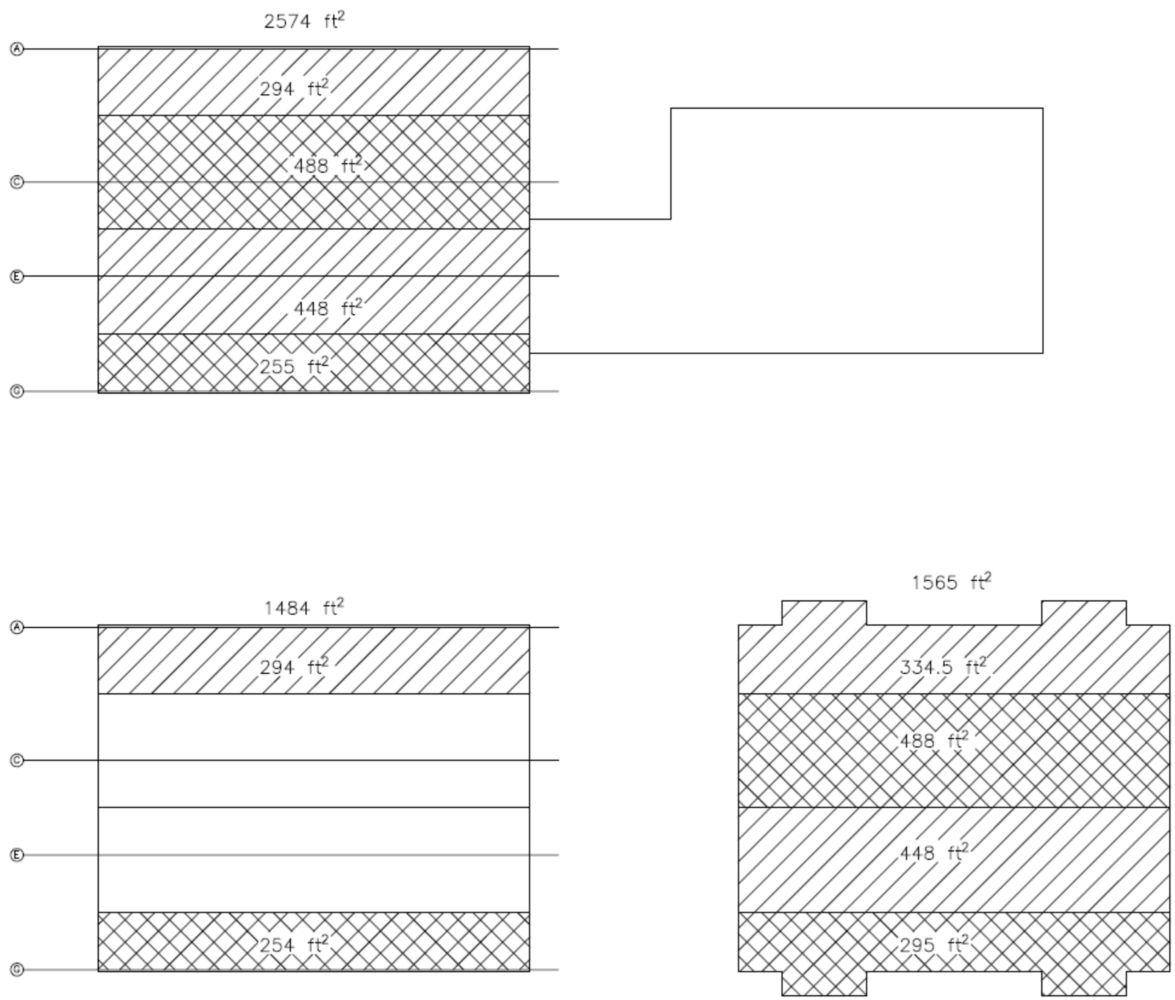


Figure B. 5. Index Bldg. 1 tributary area

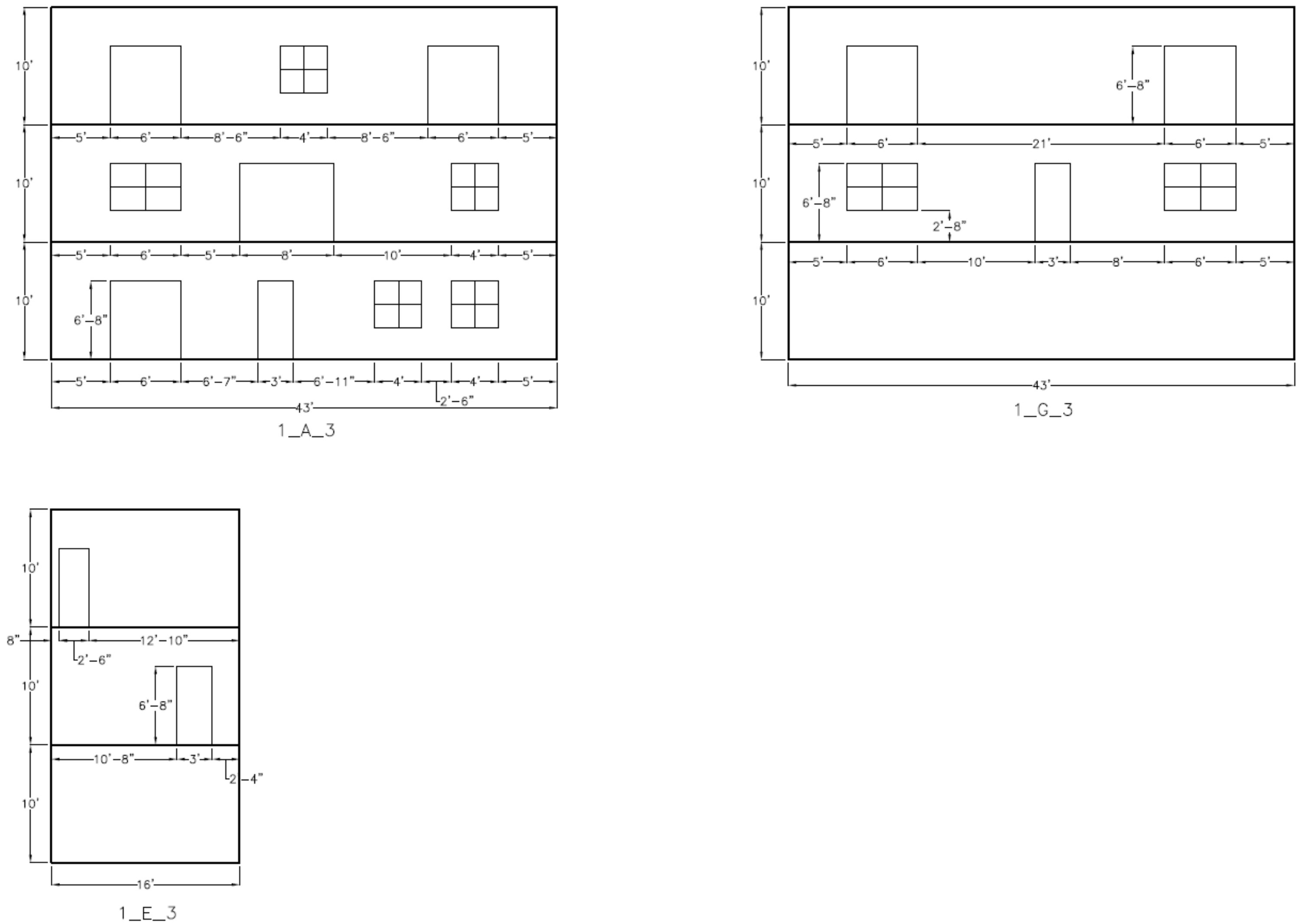


Figure B. 6. Index Bldg. 1 extracted shear wall lines

Table B. 1. Extracted Archetypes, Index Bldg. 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		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 B. 2. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 3. Seismic weight summary, Index Bldg. 1, 3 story

Level	h (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
Roof		82.0	82.0
	10		
2		114.8	141.7
	10		
1		114.2	141.1
	10		
Ground			

INPUT DATA							DESIGN SUMMARY
Total Height	h_n=	30.0	ft				Total base shear
Total Weight	W=	311	k				V = 103.67
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					

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 Bldg. 1, 3 story, R=3

INPUT DATA							DESIGN SUMMARY
Total Height	h_n=	30.0	ft				Total base shear
Total Weight	W=	365	k				V = 121.60
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					

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 Bldg. 1, 3 story, R=3

Table B. 4. Tributary load calculation, Index Bldg. 1, 3 story, E-W direction, R=3*

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)	High gravity: Story shear (kip)	High gravity: 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 B. 5. Archetype 1 (1_E_3_1_LR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	A3	3	D	1563.0	1.2	1346.9	7.82	6.73	1.16
2	1	S	5	A3	6	D	3126.0	2.3	2898.6	15.63	14.49	1.08
1	1	S	5	A3	7	D	3647.0	2.3	3485.6	18.24	17.43	1.05

Table B. 6. Archetype 13 (1_E_3_1_LR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	A3	3	D	1563.0	1.2	1304.4	7.82	6.52	1.20
2	1	S	5	A3	5	D	2605.0	1.9	2521.9	13.03	12.61	1.03
1	1	S	5	A3	6	D	3126.0	2.1	2966.6	15.63	14.83	1.05

Table B. 7. Archetype 25 (1_E_3_1_HR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	1.0	898.0	7.82	6.73	1.16
2	3	M	2.5	A3	2	D	2084.0	2.0	1932.4	15.63	14.49	1.08
1	3	M	2.5	A3	3	D	3126.0	2.6	2323.7	23.45	17.43	1.35

Table B. 8. Archetype 37 (1_E_3_1_HR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	1.0	869.6	7.82	6.52	1.20
2	3	M	2.5	A3	2	D	2084.0	1.9	1681.3	15.63	12.61	1.24
1	3	M	2.5	A3	2	D	2084.0	2.3	1977.8	15.63	14.83	1.05

Table B. 9. Archetype 97 (1_A_3_2_ HR_HG_ DX_SP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	A3	2	S	1042.0	0.4	589.3	5.21	2.95	1.77
2	2	M	2.5	A3	2	D	2084.0	0.9	1280.1	10.42	6.34	1.64
1	2	M	2.5	A3	2	D	2084.0	1.1	1736.6	10.42	8.67	1.20

*There are five of (2)2.5ft walls along the wall line and all take equal load.

Table B. 10. Archetype 109 (1_A_3_2_ HR_LG_ DX_SP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	A3	2	S	1042.0	0.4	570.7	5.21	2.85	1.83
2	2	M	2.5	A3	2	D	2084.0	0.8	1103.3	10.42	5.52	1.89
1	2	M	2.5	A3	2	D	2084.0	1.0	1486.0	10.42	7.38	1.41

*There are five of (2)2.5ft walls along the wall line and all take equal load.

INPUT DATA				DESIGN SUMMARY	
Total Height	h_n=	30.0	ft	Total base shear	
Total Weight	W=	311	k	V =	77.75
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			

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}	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

(Lateral force @ each level: F_x, V_x, O. M.)

Seismic base shear calculation, low gravity, Index Bldg. 1, 3 story, R=4

INPUT DATA				DESIGN SUMMARY	
Total Height	h_n=	30.0	ft	Total base shear	
Total Weight	W=	365	k	V =	91.20
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			

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}	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

(Lateral force @ each level: F_x, V_x, O. M.)

Seismic base shear calculation, high gravity, Index Bldg. 1, 3 story, R=4

Table B. 11. Tributary load calculation, Index Bldg. 1, 3 story, E-W direction, R=4*

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)	High gravity Story shear (kip)	High gravity 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 B. 12. Archetype 25 (1_E_3_1_HR_HG_DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	0.92	673.0	7.82	5.05	1.55
2	3	M	2.5	A3	3	S	1563.0	1.60	1448.2	11.72	10.86	1.08
1	3	M	2.5	A3	2	D	2084.0	2.03	1739.9	15.63	13.05	1.20

Table B. 13. Archetype 37 (1_E_3_1_HR_LG_DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	0.91	652.2	7.82	4.89	1.60
2	3	M	2.5	A3	3	S	1563.0	1.53	1260.9	11.72	9.46	1.24
1	3	M	2.5	A3	3	S	1563.0	1.81	1483.3	11.72	11.12	1.05

Table B. 14. Archetype 97 (1_A_3_2_ HR_HG_ DX_SP) design*, R=4

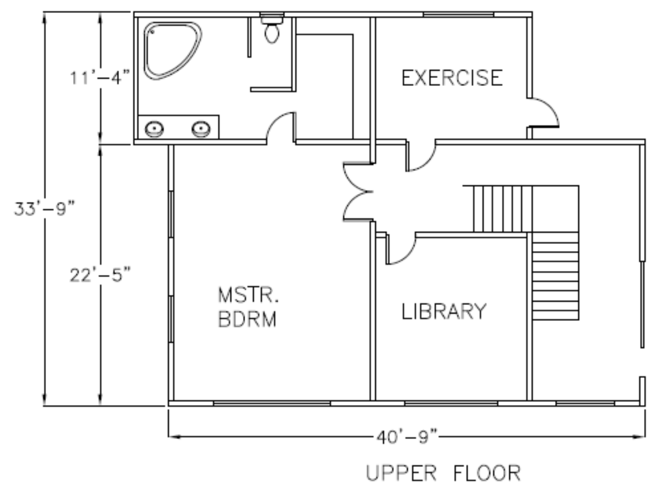
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	A3	2	S	1042.0	0.38	441.6	5.21	2.95	1.77
2	2	M	2.5	A3	2	S	1042.0	0.62	952.5	10.42	6.34	1.64
1	2	M	2.5	A3	3	S	1563.0	0.88	1298.7	10.42	8.67	1.20

*There are five of (2)2.5ft walls along the wall line and all take equal load.

Table B. 15. Archetype 109 (1_A_3_2_ HR_LG_ DX_SP) design*, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	2	M	2.5	A3	2	S	1042.0	0.37	428.0	5.21	2.85	1.83
2	2	M	2.5	A3	2	S	1042.0	0.58	827.5	10.42	5.52	1.89
1	2	M	2.5	A3	3	S	1563.0	0.80	1111.4	10.42	7.38	1.41

*There are five of (2)2.5ft walls along the wall line and all take equal load.



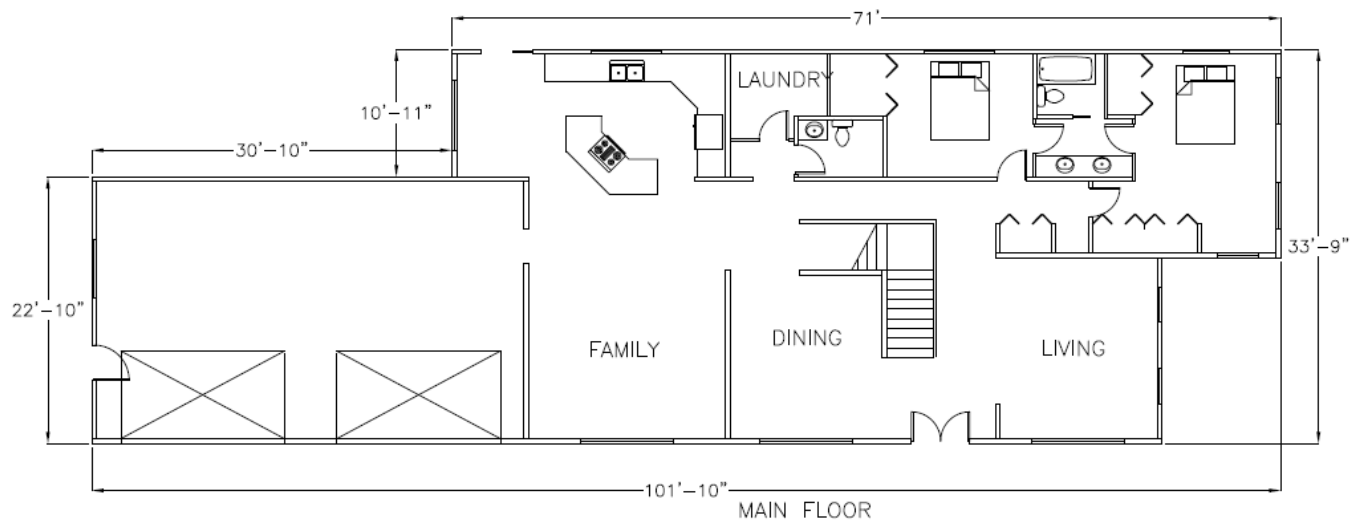


Figure B. 7. Index Bldg. 2 floor plan

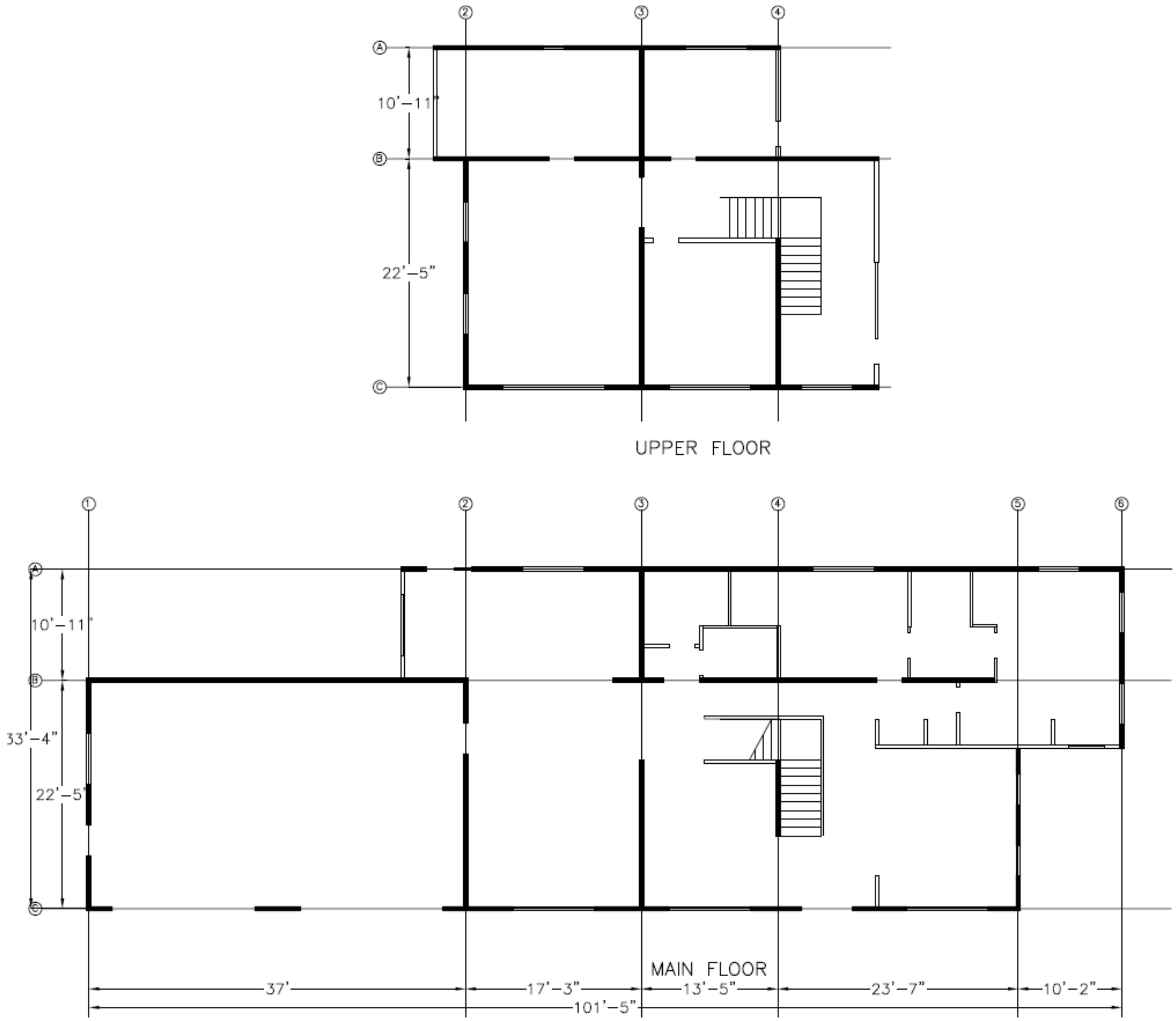


Figure B. 8. Index Bldg. 2 assigned shear walls

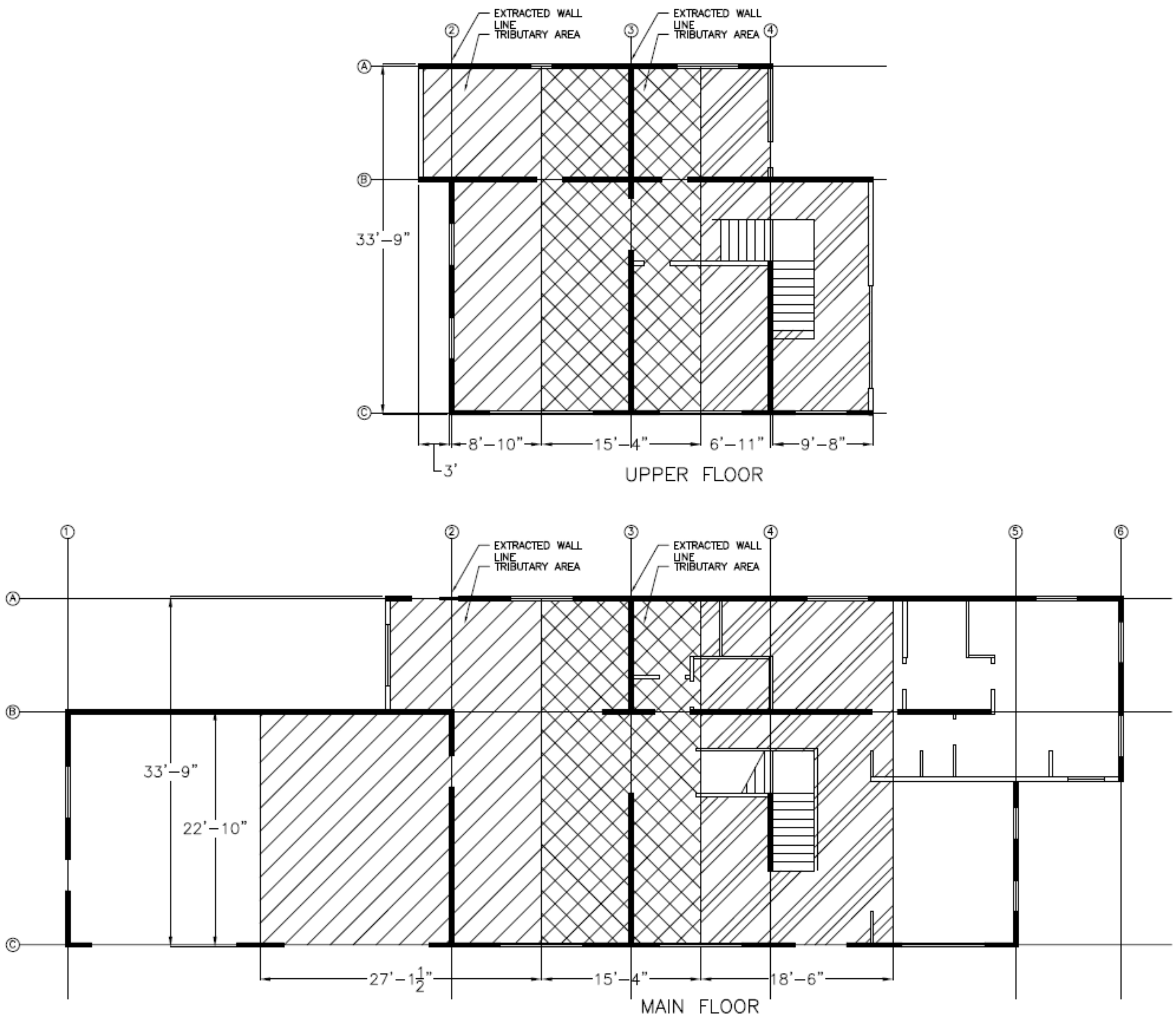


Figure B. 9. Index Bldg. 2 tributary area

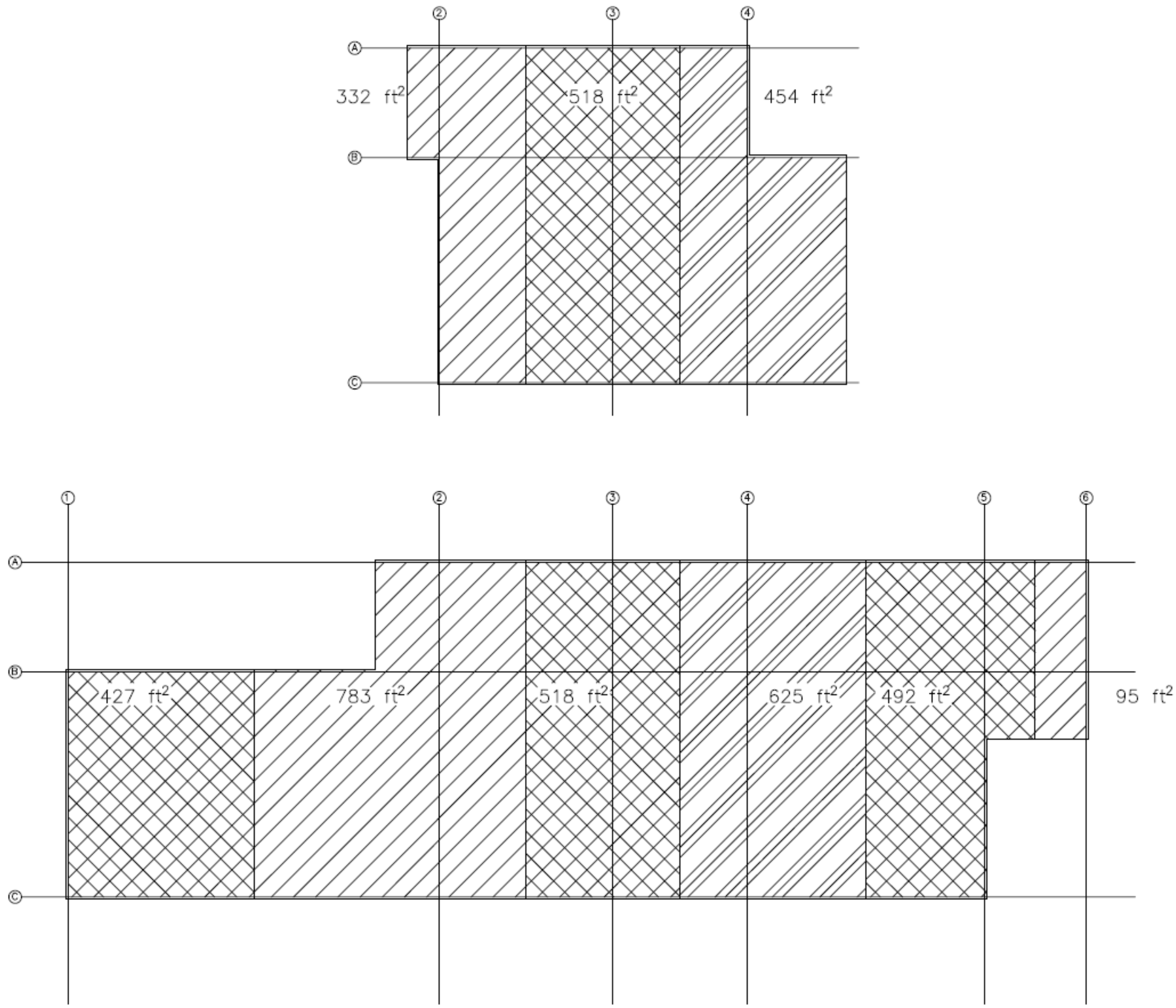


Figure B. 10. Index Bldg. 2 tributary area

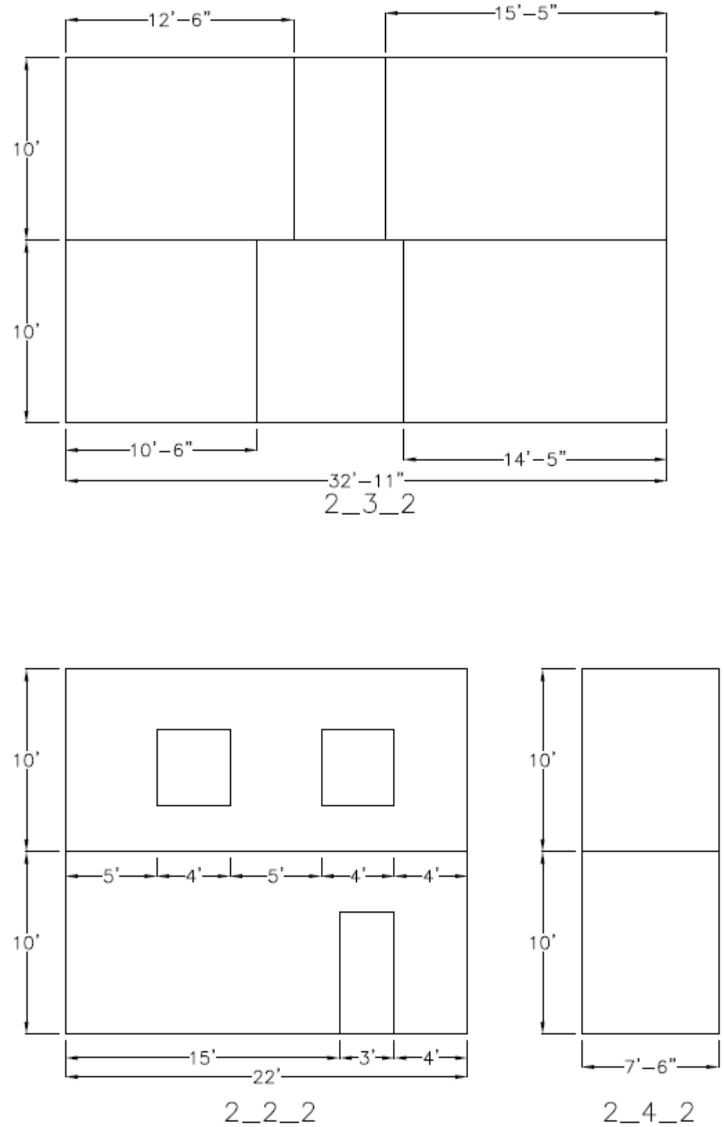


Figure B. 11. Index Bldg. 2 extracted shear wall lines

Table B. 16. Extracted Archetypes, Index Bldg. 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 B. 17. Seismic weight detailed calculation, Index Bldg. 2, 2 story

Level	Story	h (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	Low gravity: W_{floor} (lbs)	Low gravity: $W_{extWalls}$ (lbs)	Low gravity: $W_{IntWalls}$ (lbs)	Low gravity: ΣW_{Level} (lbs)	High gravity: W_{floor} (lbs)	High gravity: $W_{extWalls}$ (lbs)	High gravity: $W_{IntWalls}$ (lbs)	High gravity: ΣW_{Level} (lbs)
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 B. 18. Seismic weight summary, Index Bldg. 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				**DESIGN SUMMARY**
Total Height	h_n=	**20.0**	ft	Total base shear
Total Weight	W=	**245**	k	V = 81.53
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		

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 Bldg. 2, 2 story, R=3

INPUT DATA				**DESIGN SUMMARY**
Total Height	h_n=	**20.0**	ft	Total base shear
Total Weight	W=	**272**	k	V = 90.73
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		

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 Bldg. 2, 2 story, R=3

Table B. 19. Tributary load calculation, Index Bldg. 2, 2 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)	High gravity: Story shear (kip)	High gravity: 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 B. 20. Archetype 2 (2_4_2_1_LR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	5	D	2605	1.8	2555.5	13.025	12.78	1.02
1	1	S	5	A3	8	D	4168	2.6	3856.3	20.84	19.28	1.08

Table B. 21. Archetype 14 (2_4_2_1_LR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	5	D	2605.0	1.8	2498.3	13.03	12.49	1.04
1	1	S	5	A3	7	D	3647.0	2.5	3466.6	18.24	17.33	1.05

Table B. 22. Archetype 26 (2_4_2_1_HR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	A3	2	D	2084.0	1.7	1703.7	15.63	12.78	1.22
1	3	M	2.5	A3	3	D	3126.0	2.5	2570.9	23.45	19.28	1.22

Table B. 23. Archetype 38 (2_4_2_1_HR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	A3	2	D	2084.0	1.7	1665.5	15.63	12.49	1.25
1	3	M	2.5	A3	3	D	3126.0	2.3	2311.0	23.45	17.33	1.35

Table B. 24. Archetype 73 (2_2_2_2_ LR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.8	934.4	-	-	-
1	1	S	5	A3	5	D	2605.0	1.7	2415.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.81	934.4	10.42	9.34	1.12
1	1	S	5	A3	5	D	2605.0	1.69	2415.6	26.05	24.16	1.08

Table B. 25. Archetype 85 (2_2_2_2_ LR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.8	913.5	-	-	-
1	1	S	5	A3	5	D	2605.0	1.6	2171.4	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.80	913.5	10.42	9.13	1.14
1	1	S	5	A3	5	D	2605.0	1.62	2171.4	26.05	21.71	1.20

Table B. 26. Archetype 121 (2_2_2_2_ MR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.5	617.9	-	-	-
1	2	M	2.5	A3	2	D	2084.0	0.9	1434.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	5	S	1302.5	0.93	1250.9	11.72	9.34	1.25
1	1	S	5	A3	7	D	3647.0	2.23	3396.4	28.66	24.16	1.19

Table B. 27. Archetype 133 (2_2_2_2_ MR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.5	715.6	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.0	1549.5	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	5	S	1302.5	0.76	1111.3	11.72	9.13	1.28
1	1	S	5	A3	6	D	3126.0	1.74	2793.4	26.05	21.71	1.20

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 Bldg. 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 Bldg. 2, 2 story, R=4

Table B. 28. Tributary load calculation, Index Bldg. 2, 2 story, N-S direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)	High gravity: Story shear (kip)	High gravity: 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 B. 29. Archetype 26 (2_4_2_1_HR_HG_DX_SP) design

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	A3	3	S	1563.0	1.40	1276.6	11.72	9.57	1.22
1	3	M	2.5	A3	2	D	2084.0	1.98	1930.3	15.63	14.48	1.08

Table B. 30. Archetype 38 (2_4_2_1_HR_LG_DX_SP) design

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	3	M	2.5	A3	3	S	1563.0	1.39	1249.2	11.72	9.37	1.25
1	3	M	2.5	A3	4	S	2084.0	1.92	1733.3	15.63	13.00	1.20

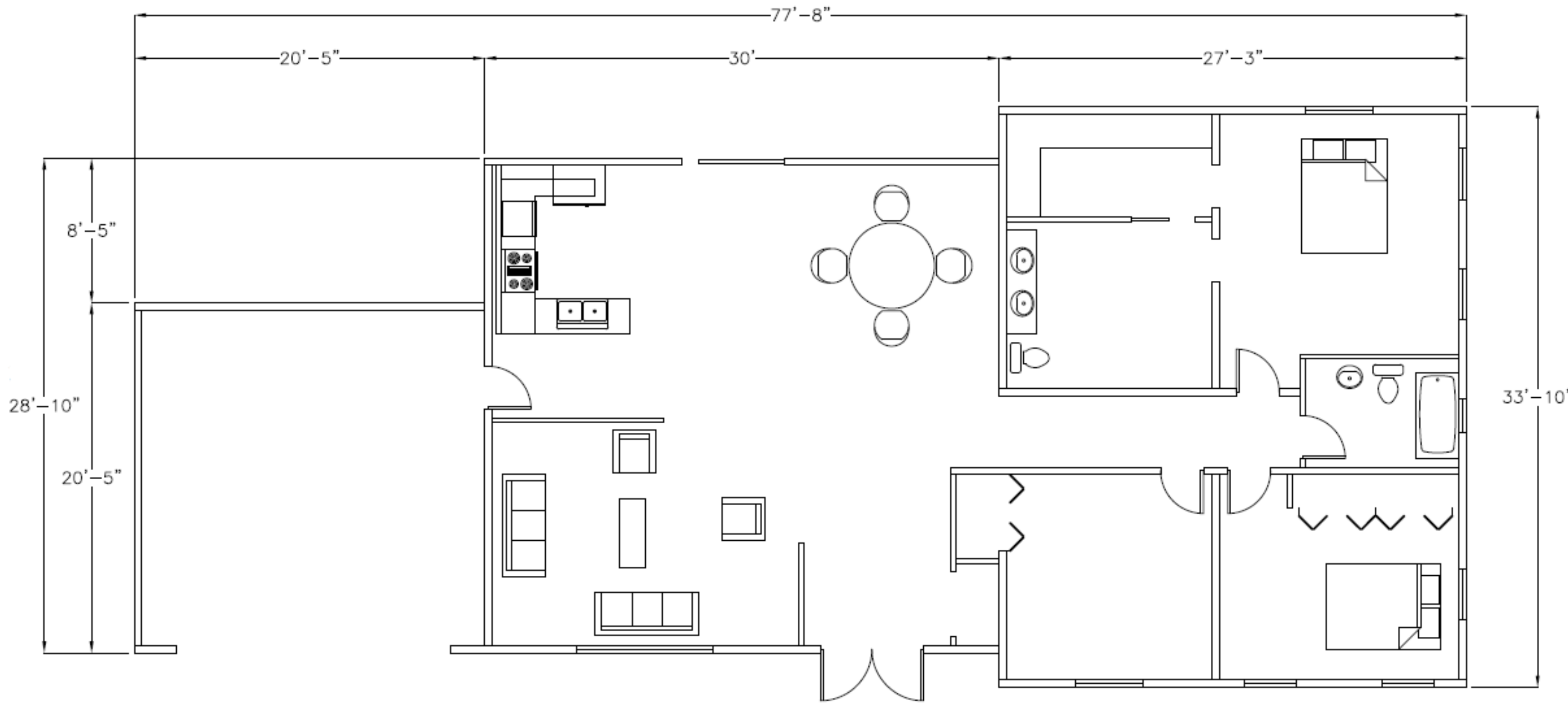


Figure B. 12. Index Bldg. 3 floor plan

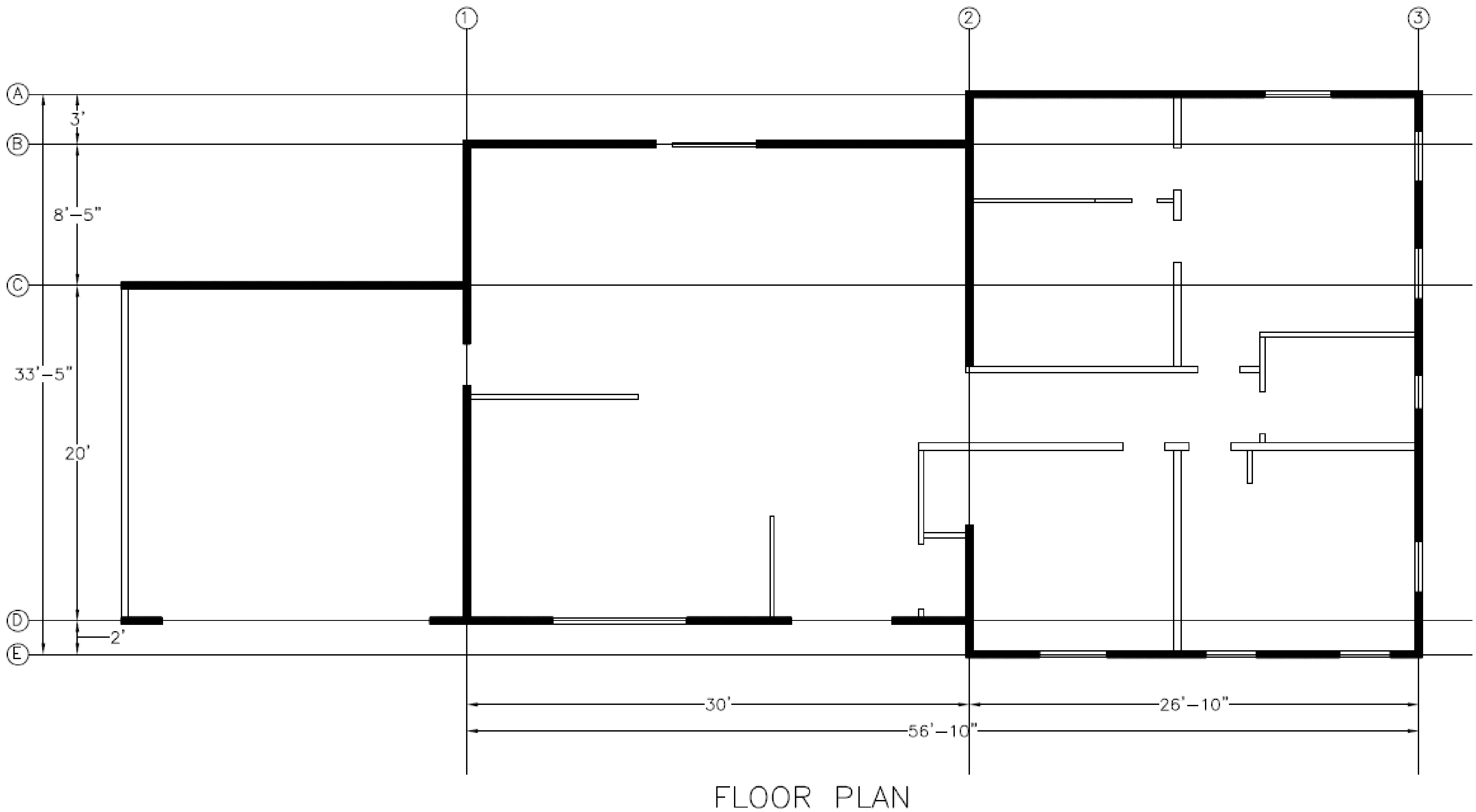


Figure B. 13. Index Bldg. 3 assigned shear walls

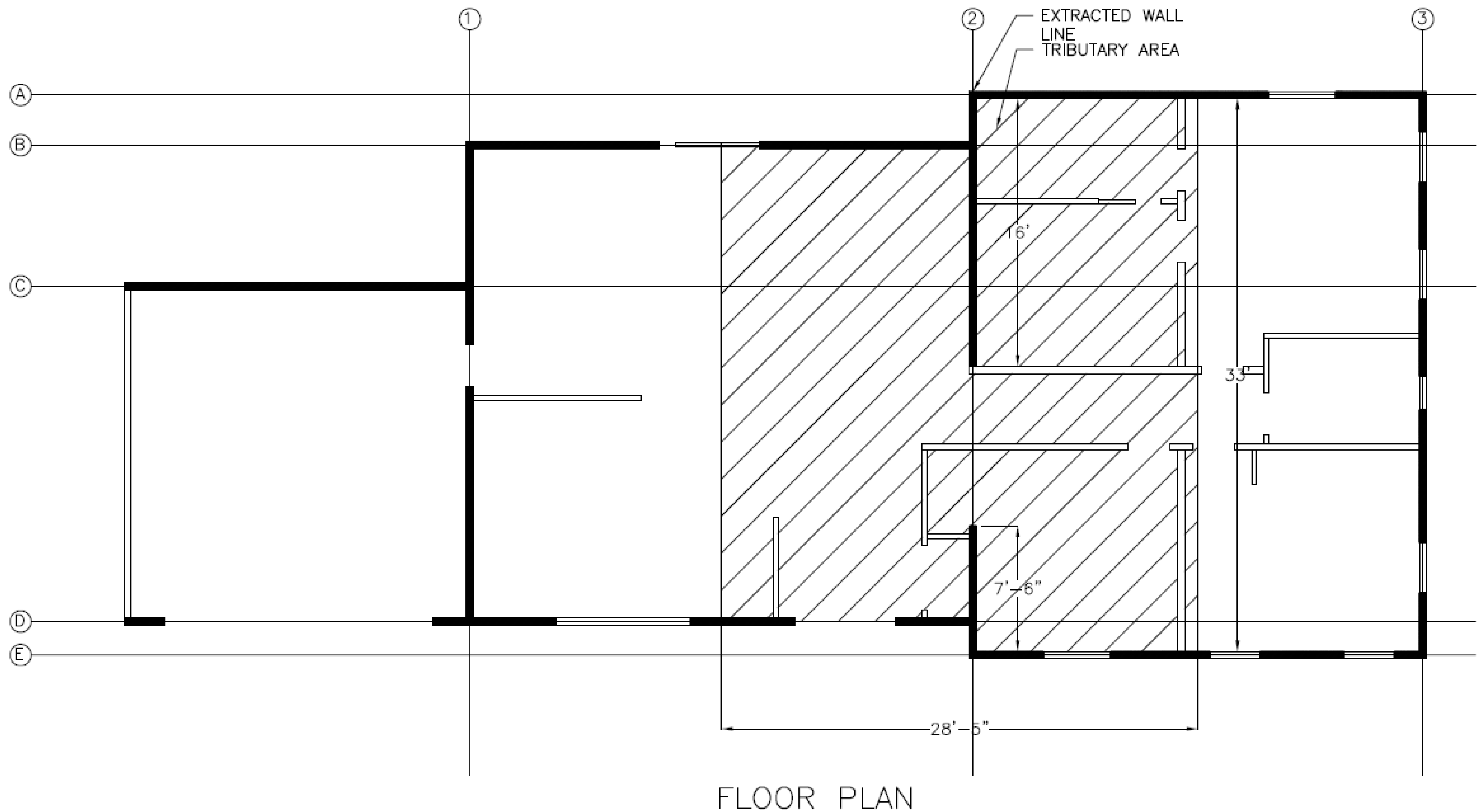


Figure B. 14. Index Bldg. 3 tributary area

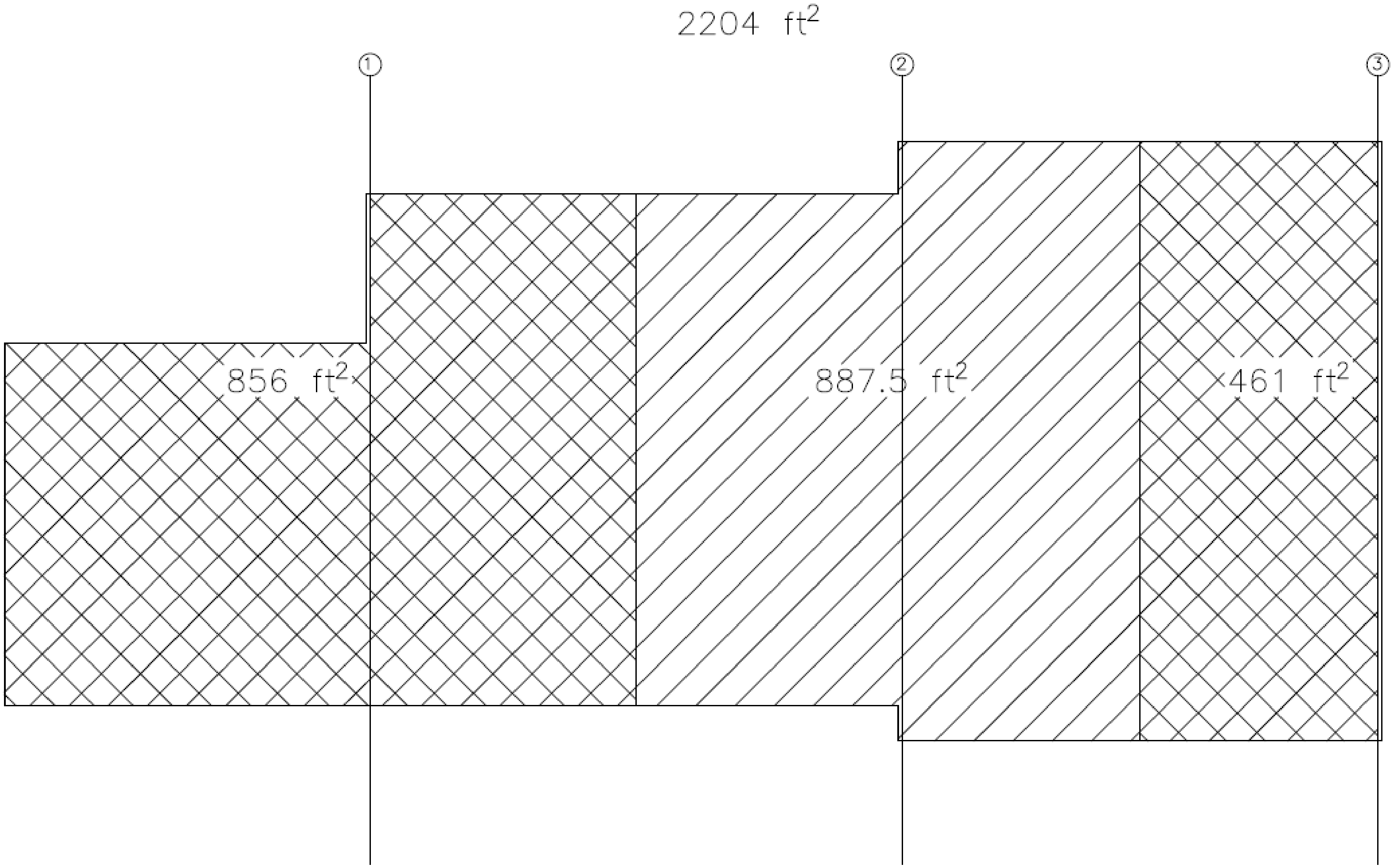


Figure B. 15. Index Bldg. 3 tributary area

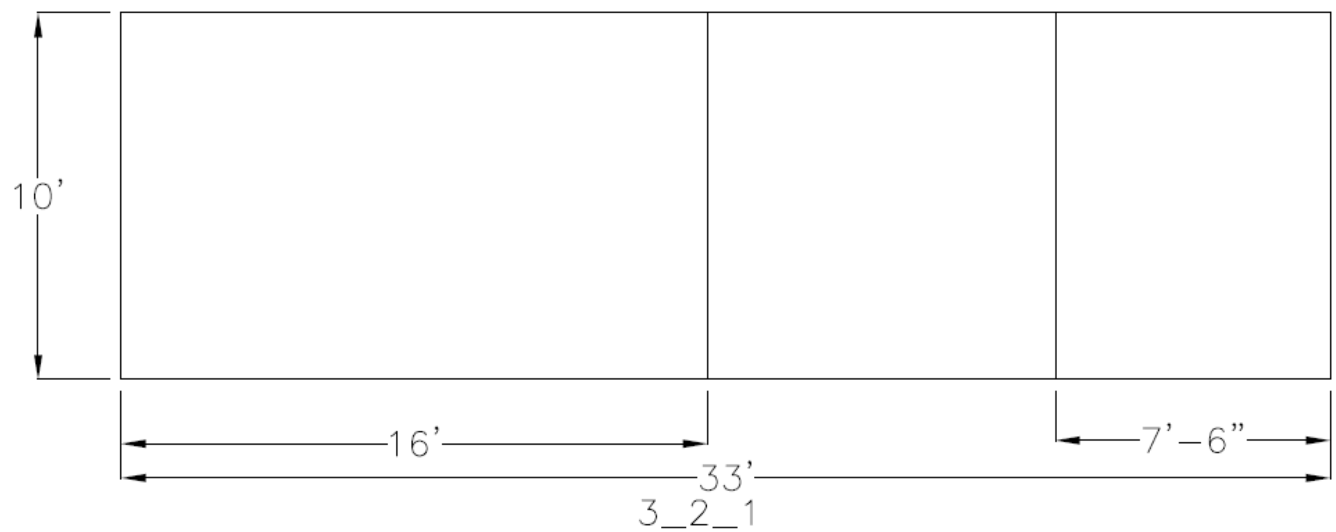


Figure B. 16. Index Bldg. 3 extracted shear wall line

Table B. 31. Extracted Archetypes, Index Bldg. 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			Low		3_2_1_2_ LR_LG_ DX_SP	86
PG-41		Mixed aspect ratio	High		3_2_1_2_ MR_HG_ DX_SP	122
PG-45			Low		3_2_1_2_ MR_LG_ DX_SP	134

Table B. 32. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
Roof			2204					64503.7			109899	64503.7			109899
	6	10		221.3	1898.7	113.21	1032		64025	26765			64025	26765	
Ground			0												

Table B. 33. Seismic weight summary, Index Bldg. 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 Bldg. 3, 1 story, R=3

Table B. 34. Tributary load calculation, Index Bldg. 3, 1 story , N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)	High gravity: Story shear (kip)	High gravity: 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 B. 35. Archetype 74 (3_2_1_2_ LR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	A3	4	S	1042.0	4.4	914.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	1	S	5	A3	2	S	521.0	0.32	202.8	18.24	14.73	1.24

Table B. 36. Archetype 86 (3_2_1_2_ LR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	S	5	A	4	S	1042.0	0.90	982.3	15.63	14.73	1.06

Table B. 37. Archetype 122 (3_2_1_2_ MR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	2.5	A3	2	S	1042.0	0.5	200.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	3	M	5	A3	4	S	1042.0	4.50	882.0	23.45	14.73	1.59

Table B. 38. Archetype 134 (3_2_1_2_ MR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	1	S	5	A	5	D	2605.0	1.9	2334.1	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
1	2	M	2.5	A	2	S	1042.0	0.50	612.8	18.24	14.73	1.24

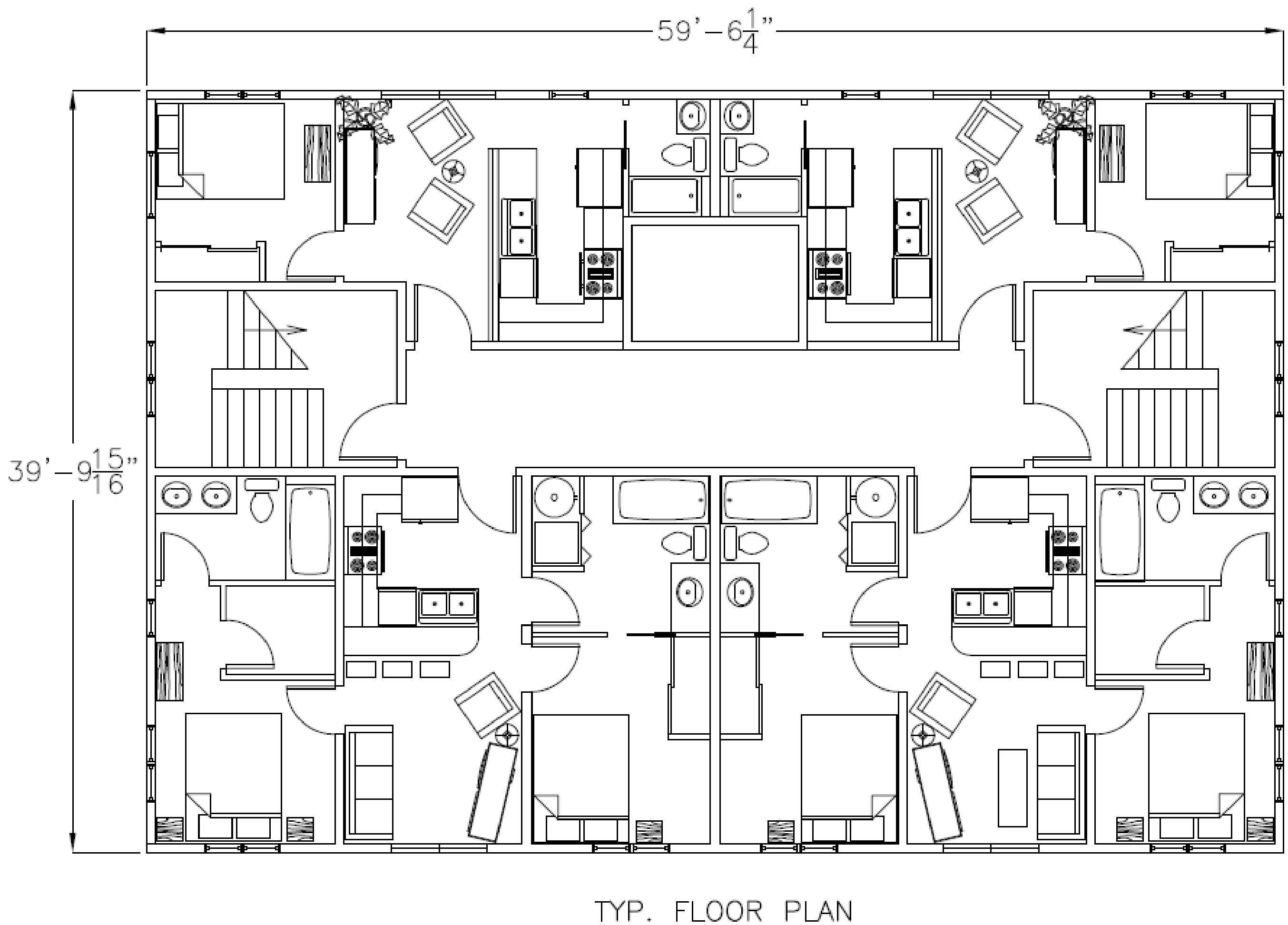


Figure B. 17. Index Bldg. 4 floor plan

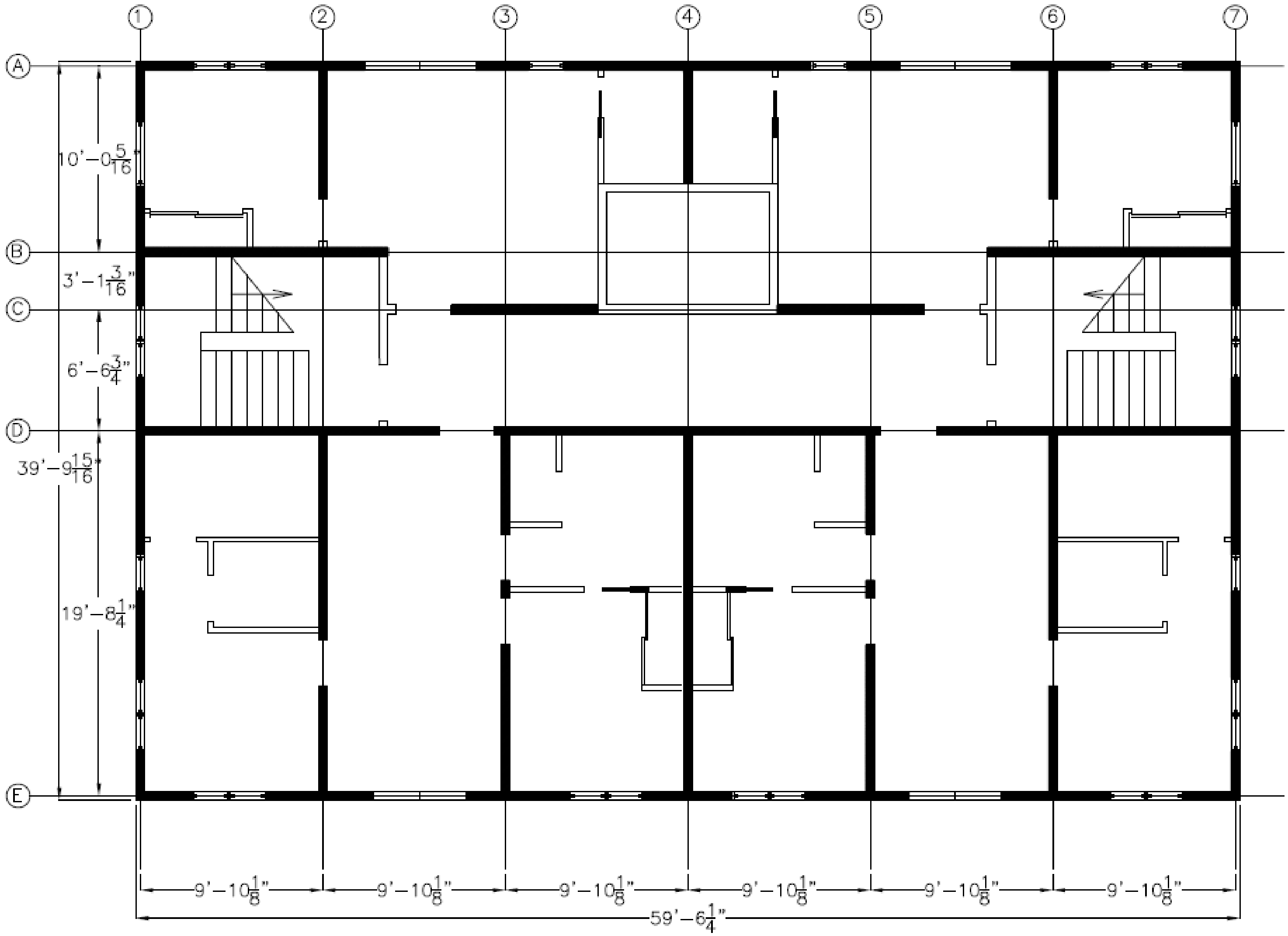


Figure B. 18. Index Bldg. 4 assigned shear walls

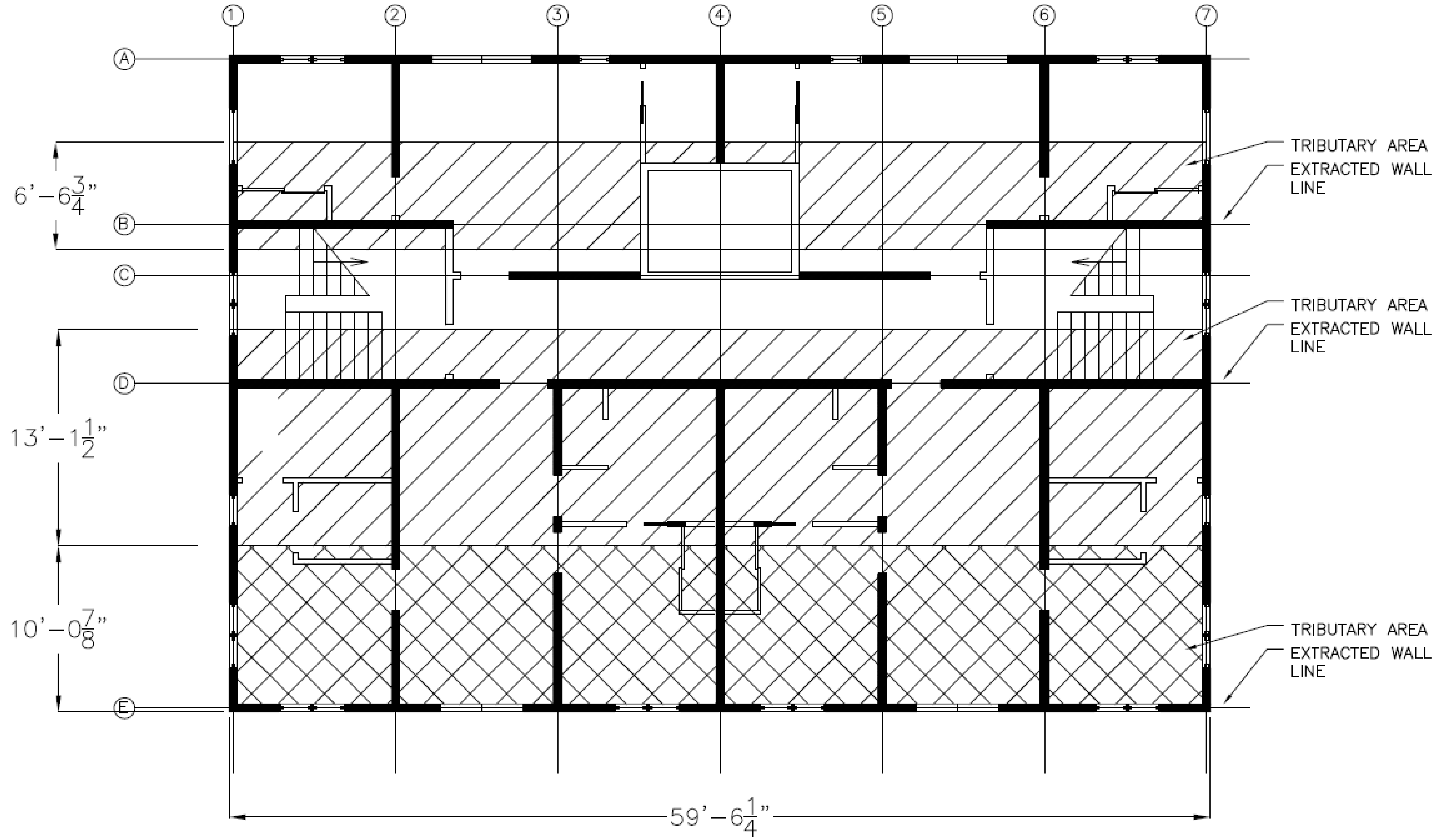


Figure B. 19. Index Bldg. 4 tributary area

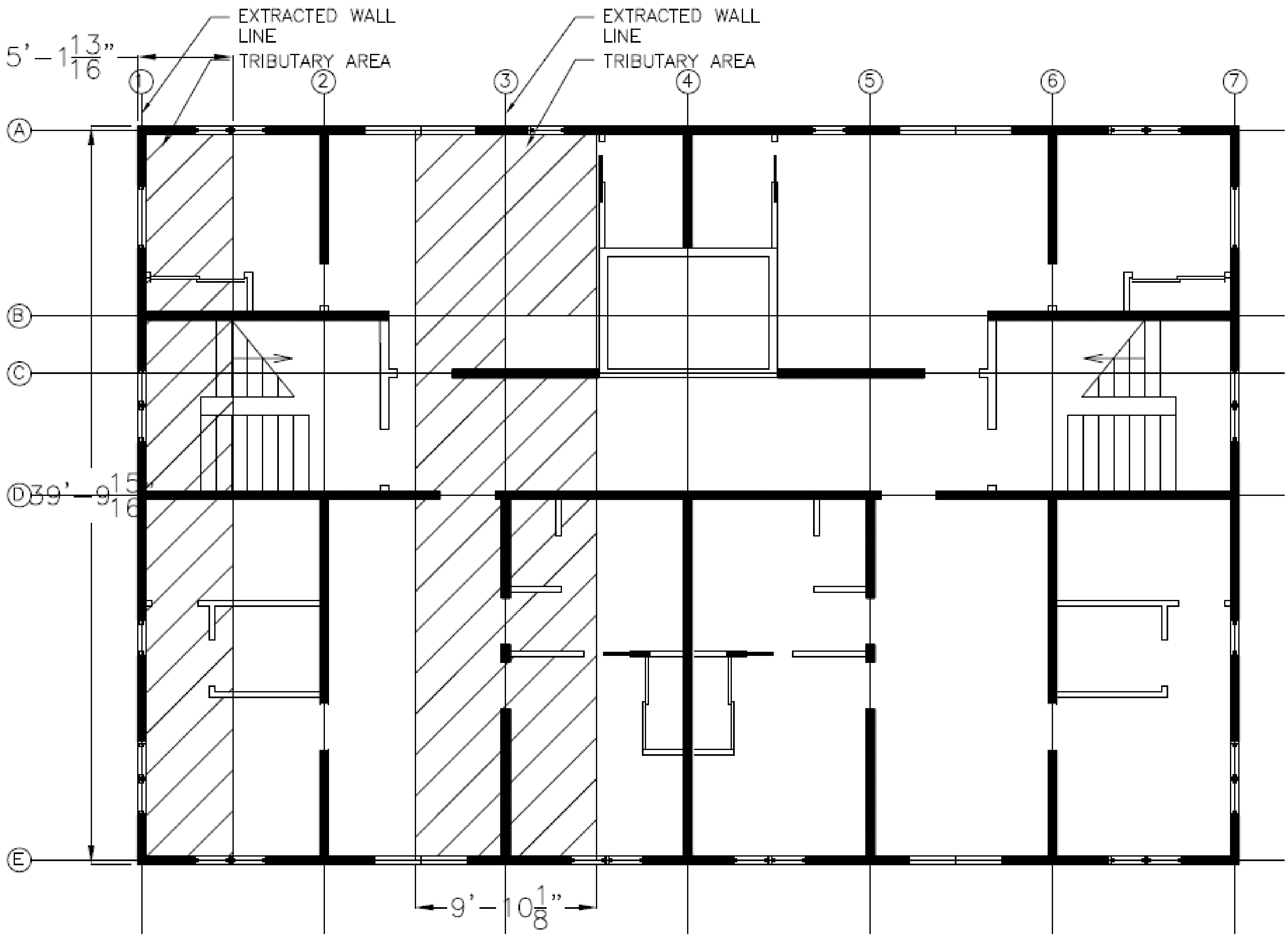


Figure B. 20. Index Bldg. 4 tributary area

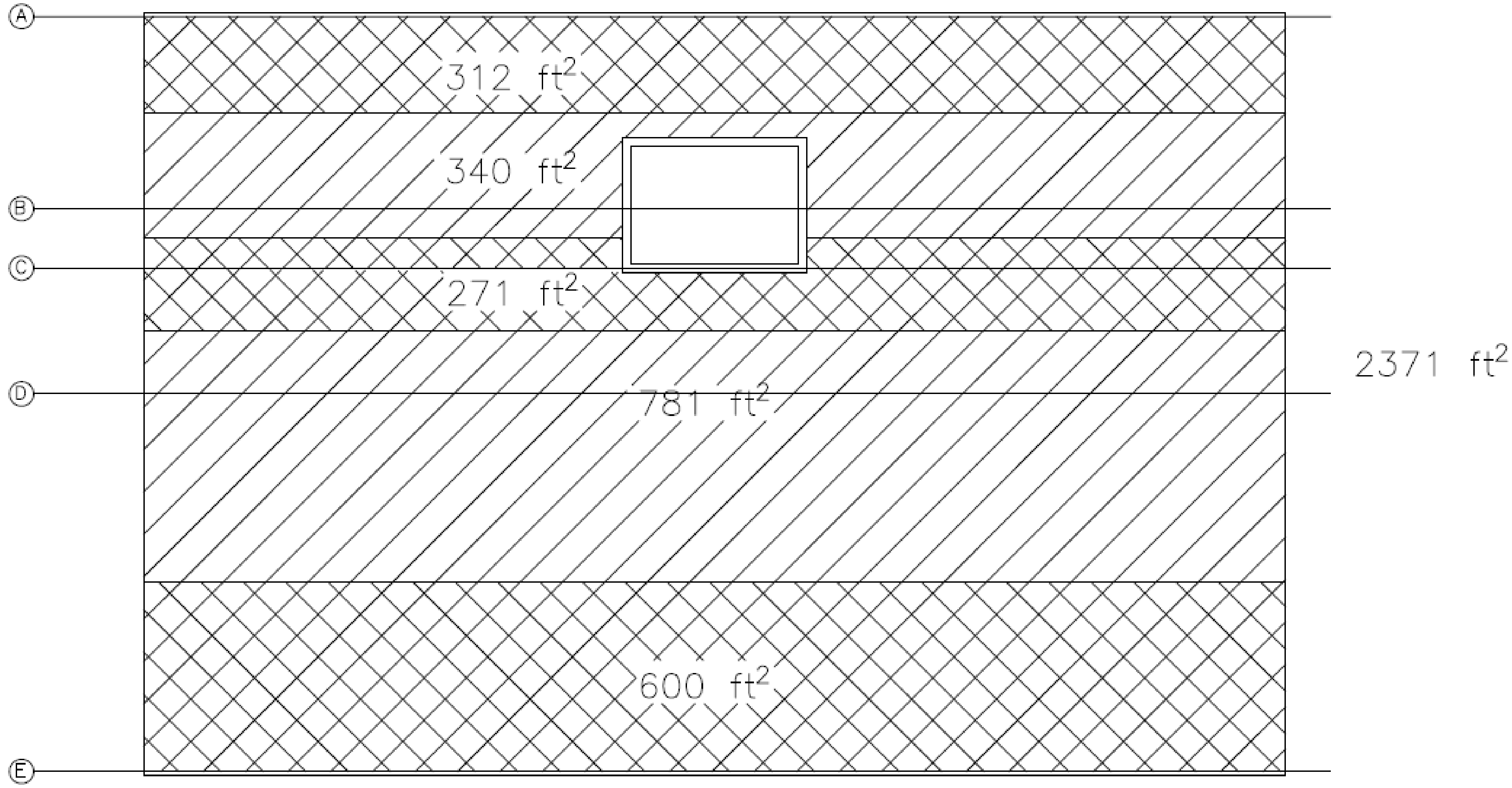


Figure B. 21. Index Bldg. 4 tributary area

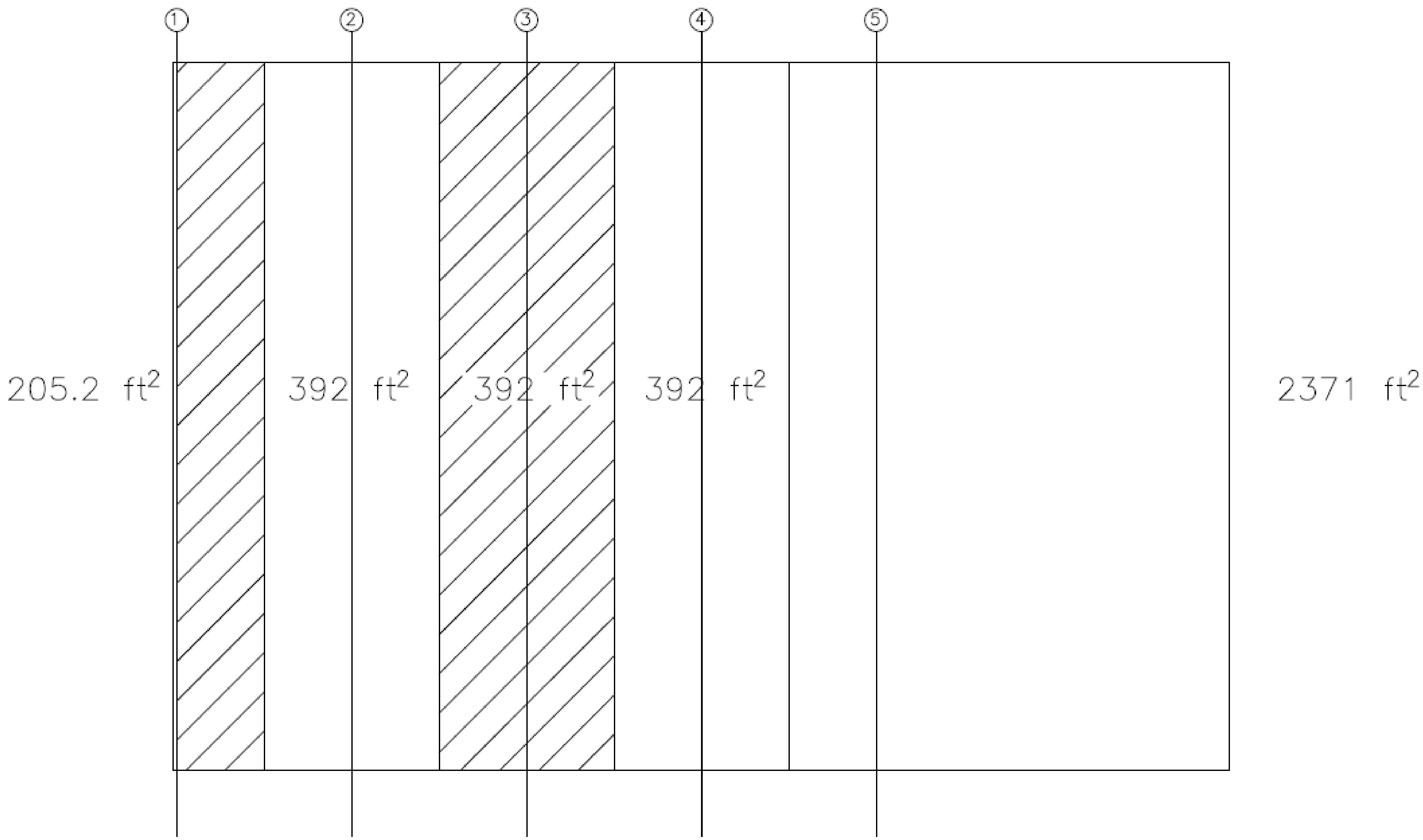


Figure B. 22. Index Bldg. 4 tributary area

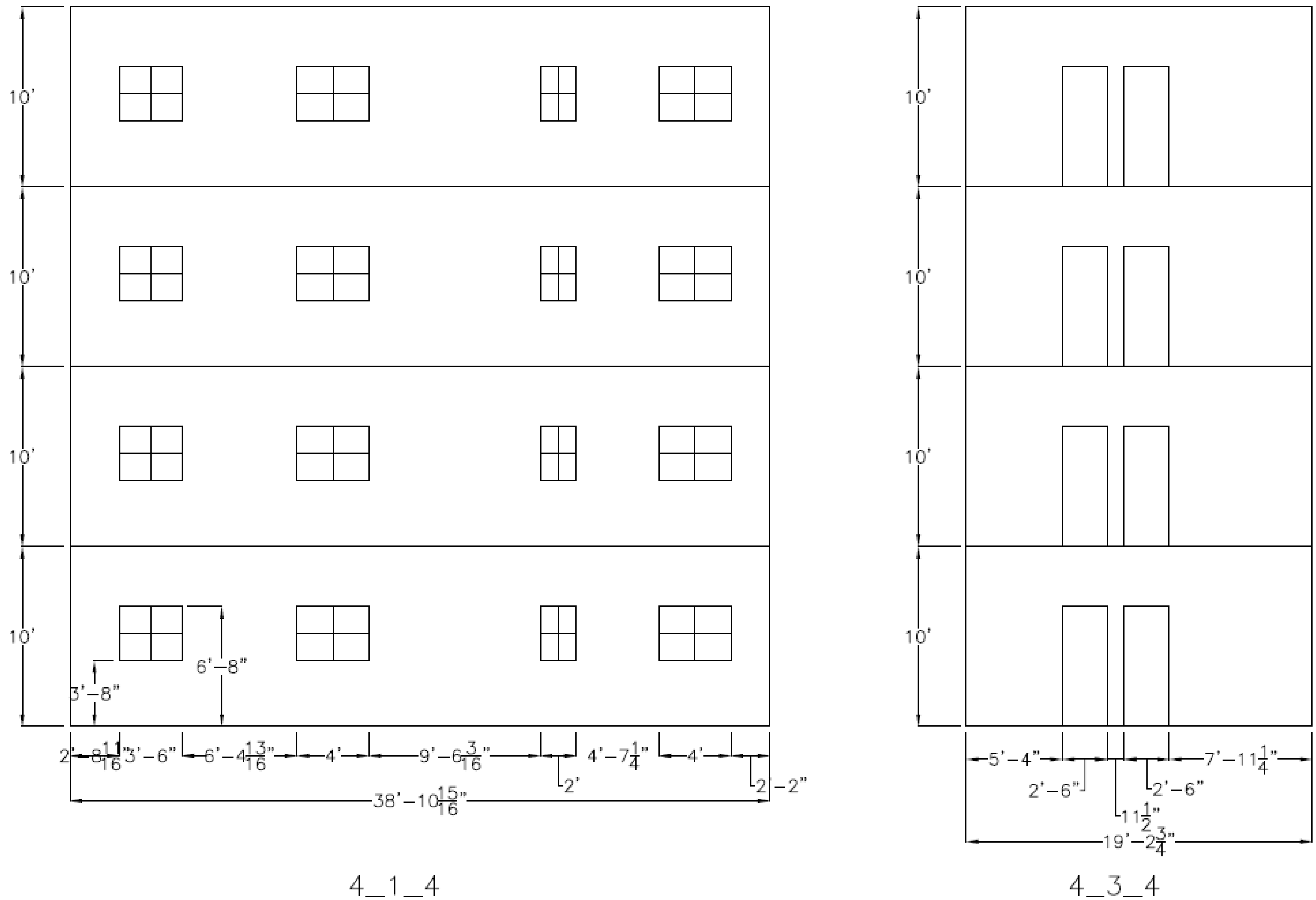


Figure B. 23. Index Bldg. 4 extracted shear wall lines

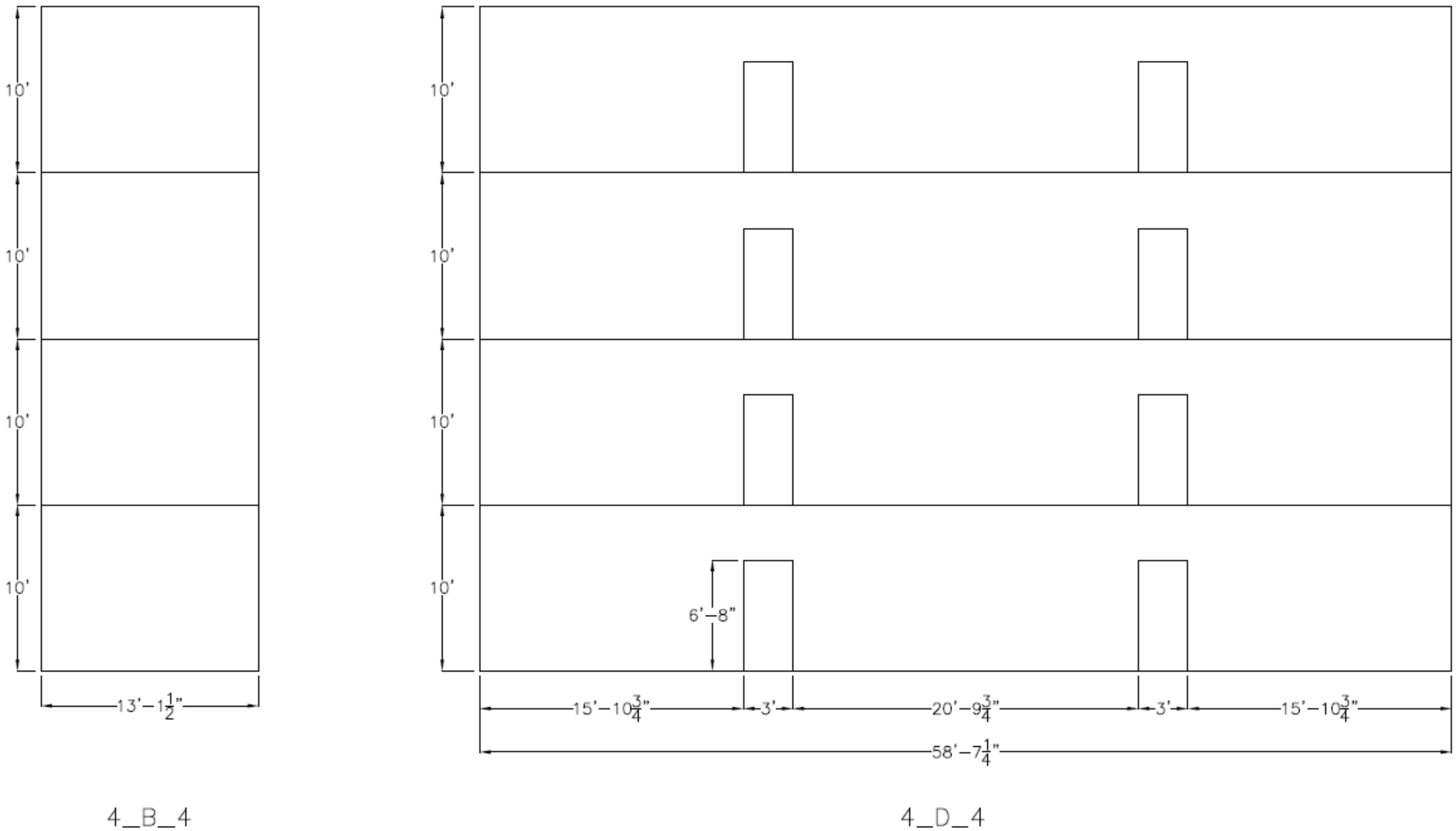


Figure B. 24. Index Bldg. 4 extracted shear wall lines

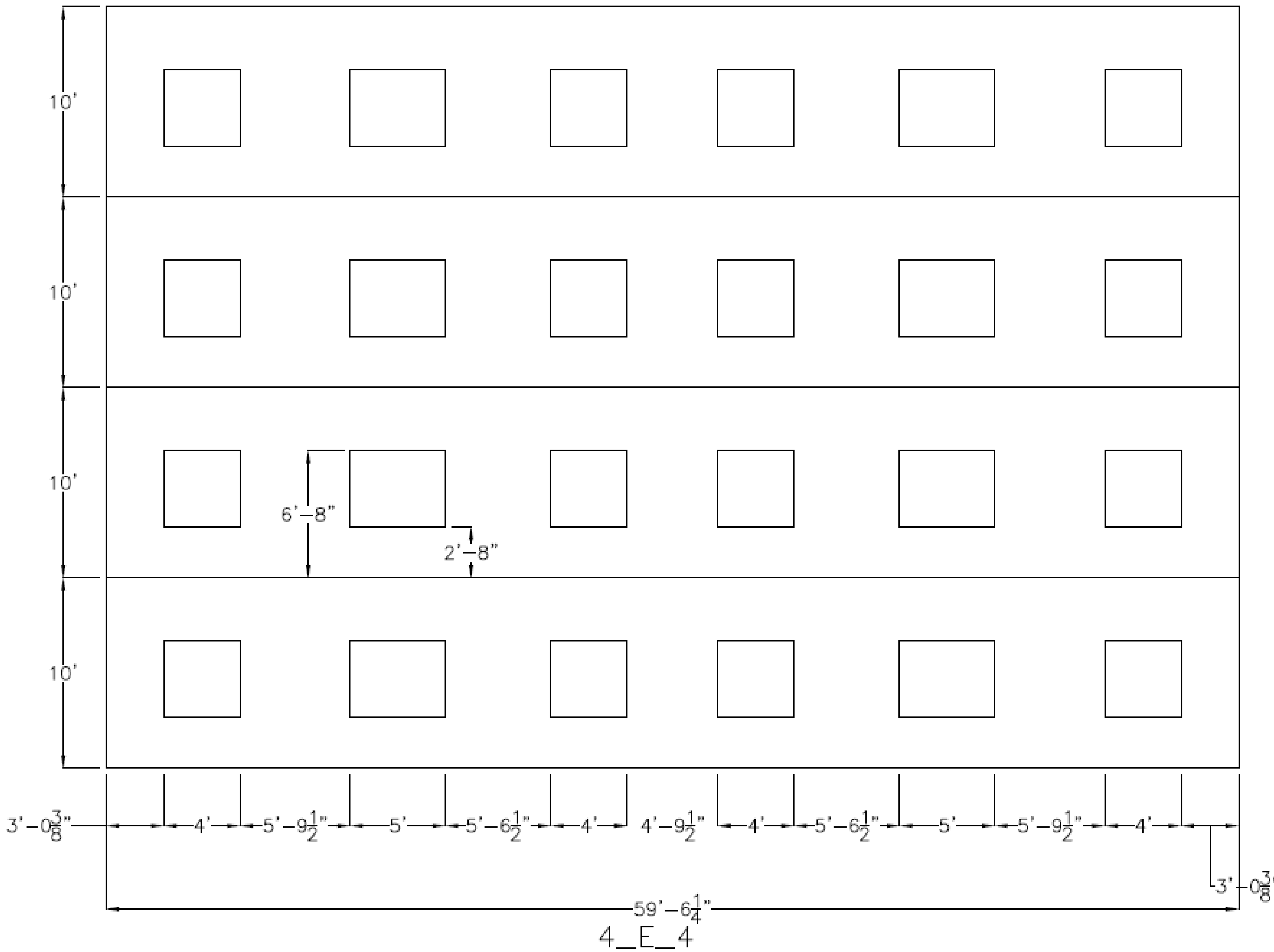


Figure B. 25. Index Bldg. 4 extracted shear wall lines

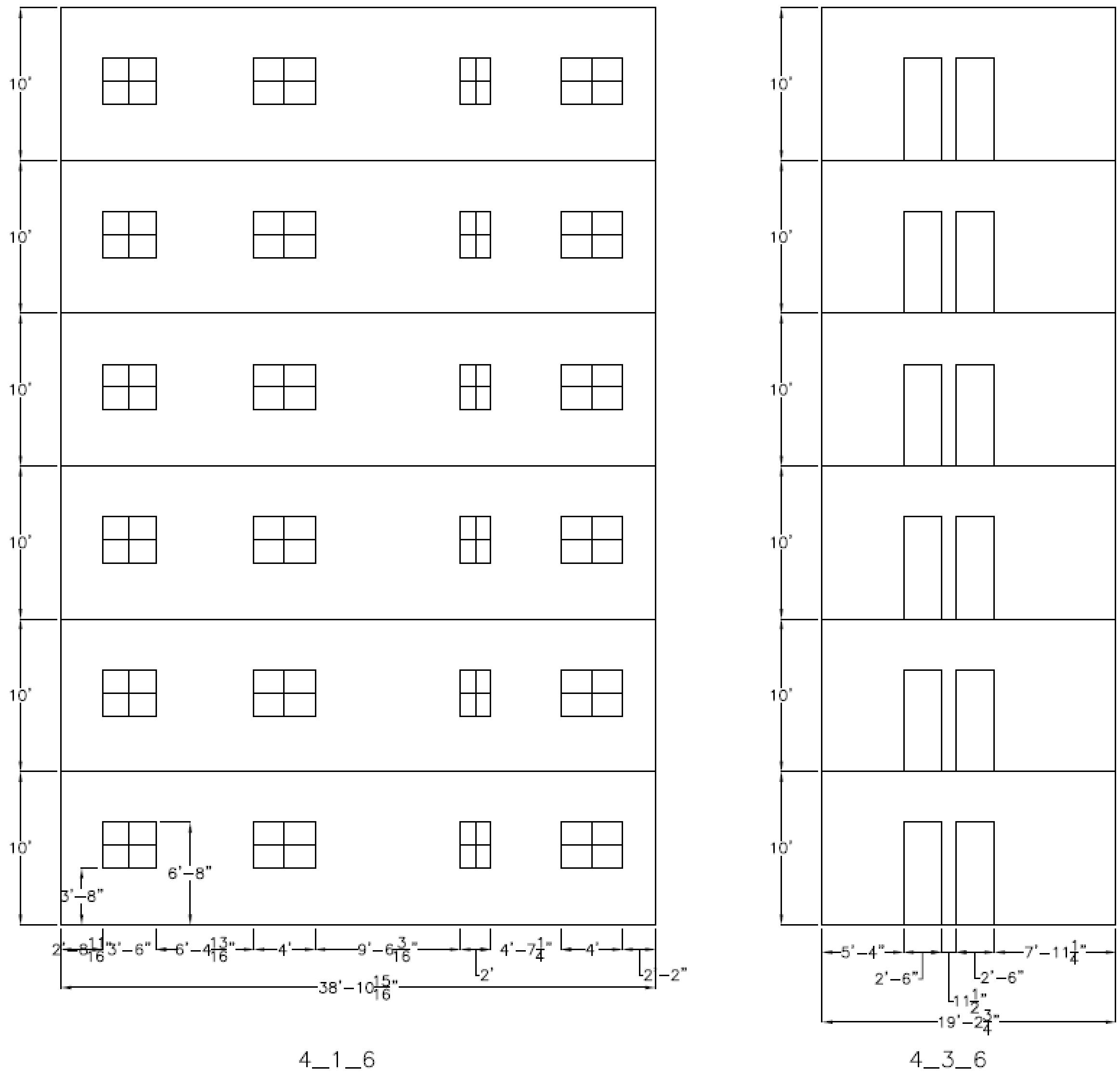


Figure B. 26. Index Bldg. 4 extracted shear wall lines

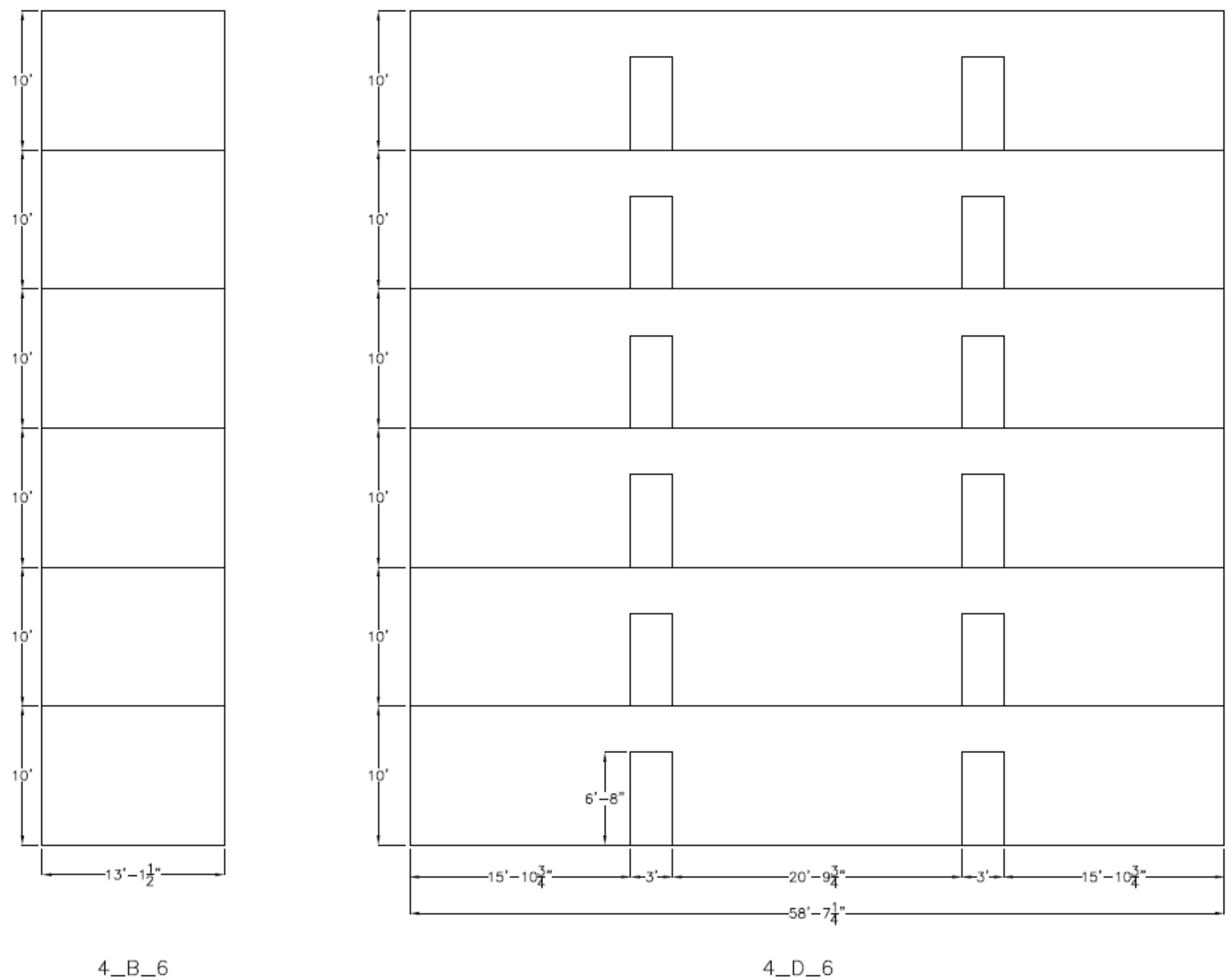


Figure B. 27. Index Bldg. 4 extracted shear wall lines

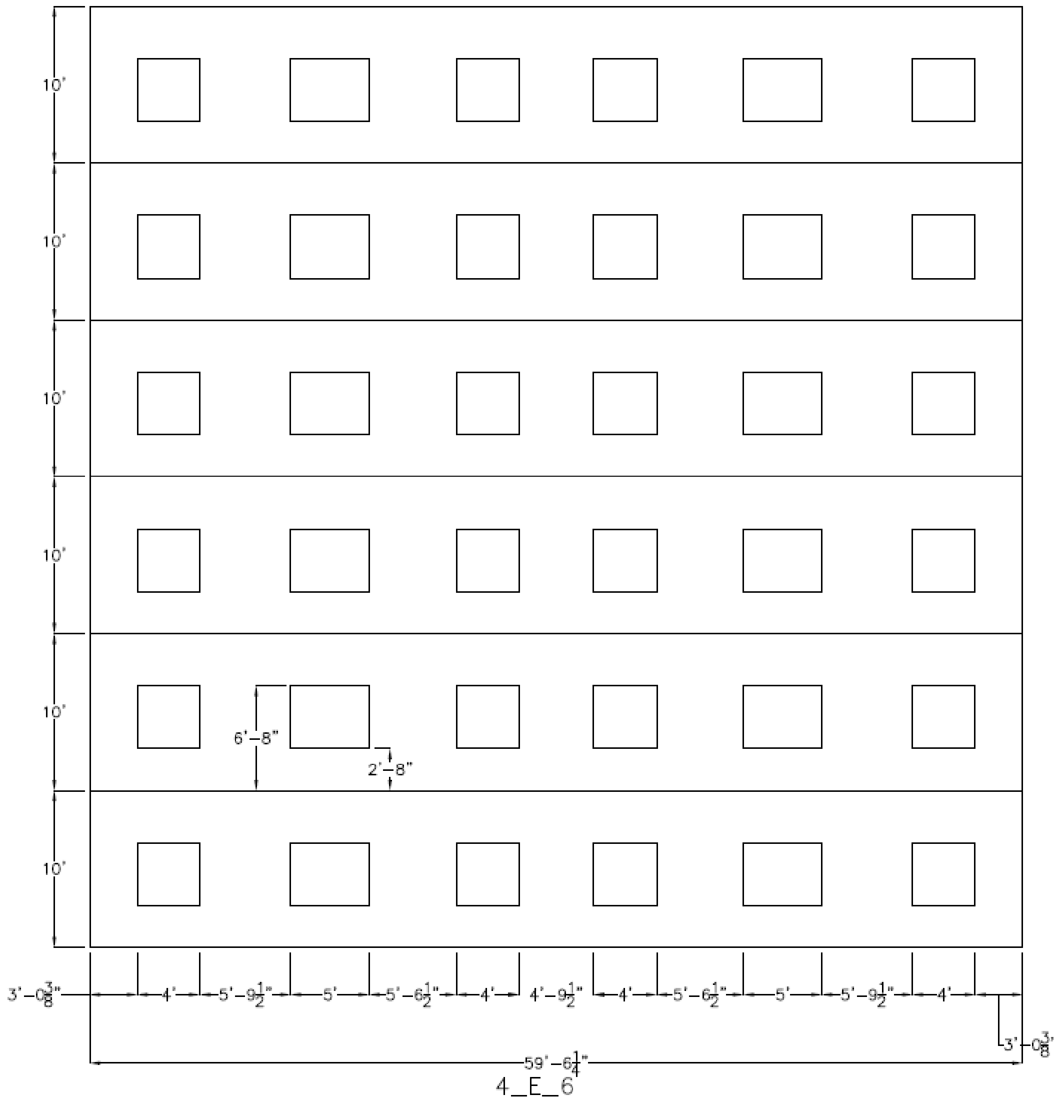


Figure B. 28. Index Bldg. 4 extracted shear wall lines

Table B. 39. Extracted Archetypes, Index Bldg. 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 B. 40. Seismic weight detailed calculation, Index Bldg. 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} (lbs)**	**$W_{extWalls}$ (lbs)**	**$W_{IntWalls}$ (lbs)**	**ΣW_{Level} (lbs)**	**W_{floor} (lbs)**	**$W_{extWalls}$ (lbs)**	**$W_{IntWalls}$ (lbs)**	**ΣW_{Level} (lbs)**
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 B. 41. Seismic weight detailed calculation, Index Bldg. 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} (lbs)**	**$W_{extWalls}$ (lbs)**	**$W_{IntWalls}$ (lbs)**	**ΣW_{Level} (lbs)**	**W_{floor} (lbs)**	**$W_{extWalls}$ (lbs)**	**$W_{IntWalls}$ (lbs)**	**ΣW_{Level} (lbs)**
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 B. 42. Seismic weight summary, Index Bldg. 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 B. 43. Seismic weight summary, Index Bldg. 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}	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

(Lateral force @ each level: F_x, V_x, O. M.)

Seismic base shear calculation, low gravity, Index Bldg. 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}	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

(Lateral force @ each level: F_x, V_x, O. M.)

Seismic base shear calculation, high gravity, Index Bldg. 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 Bldg. 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 Bldg. 4, 6 story, R=3

Table B. 44. Tributary load calculation, Index Bldg. 4, 4 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity		High gravity	
				Story shear (kip)	Cumulative Shear Load (kip)	Story shear (kip)	Cumulative Shear Load (kip)
Story 4	1	205.0	0.086	70.60	6.11	72.37	6.26
	2	392.0	0.165		11.68		11.97
	3	**392.0**	**0.165**		**11.68**		**11.97**
	4	392.0	0.165		11.68		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
Story 3	1	205.0	0.086	149.12	12.90	171.46	14.83
	2	392.0	0.165		24.67		28.36
	3	**392.0**	**0.165**		**24.67**		**28.36**
	4	392.0	0.165		24.67		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
Story 2	1	205.0	0.086	201.47	17.43	237.52	20.55
	2	392.0	0.165		33.32		39.29
	3	**392.0**	**0.165**		**33.32**		**39.29**
	4	392.0	0.165		33.32		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
Story 1	1	205.0	0.086	227.97	19.72	270.90	23.43
	2	392.0	0.165		37.71		44.81
	3	**392.0**	**0.165**		**37.71**		**44.81**
	4	392.0	0.165		37.71		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 B. 45. Tributary load calculation, Index Bldg. 4, 6 story, N-S direction, R=3*

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 46. Tributary load calculation, Index Bldg. 4, 6 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 47. Archetype 4 (4_3_ 6_1_LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.5	640.1	-	-	-
5	1	S	5	A3	4	D	2084.0	1.4	1604.7	-	-	-
4	1	S	5	A3	5	D	2605.0	1.8	2367.6	-	-	-
3	1	S	5	A3	6	D	3126.0	2.3	2948.6	-	-	-
2	1	S	5	A3	7	D	3647.0	2.4	3275.6	-	-	-
1	1	S	5	A3	7	D	3647.0	2.6	3463.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.55	640.1	7.82	6.40	1.22
5	1	S	5	A3	4	D	2084.0	1.39	1604.7	20.84	16.05	1.30
4	1	S	5	A3	5	D	2605.0	1.80	2367.6	26.05	23.68	1.10
3	1	S	5	A3	6	D	3126.0	2.27	2913.9	31.26	29.31	1.07
2	1	S	5	A3	7	D	3647.0	2.48	3322.8	36.47	32.99	1.11
1	1	S	5	A3	7	D	3647.0	2.62	3493.9	36.47	34.79	1.05

Table B. 48. Archetype 16 (4_3_ 6_1_LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.5	628.1	-	-	-
5	1	S	5	A3	3	D	1563.0	1.2	1396.9	-	-	-
4	1	S	5	A3	4	D	2084.0	1.6	2004.8	-	-	-
3	1	S	5	A3	5	D	2605.0	1.8	2454.1	-	-	-
2	1	S	5	A3	6	D	3126.0	2.1	2747.3	-	-	-
1	1	S	5	A3	6	D	3126.0	2.1	2890.5	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.54	628.1	7.82	6.28	1.24
5	1	S	5	A3	3	D	1563.0	1.16	1396.9	15.63	13.97	1.12
4	1	S	5	A3	4	D	2084.0	1.61	2004.8	20.84	20.05	1.04
3	1	S	5	A3	5	D	2605.0	1.83	2454.1	26.05	24.54	1.06
2	1	S	5	A3	6	D	3126.0	2.10	2747.3	31.26	27.47	1.14
1	1	S	5	A3	6	D	3126.0	2.06	2890.5	31.26	28.91	1.08

Table B. 49. Archetype 28 (4_3_ 6_1_HR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.9	621.5	-	-	-
5	3	M	2.5	A3	2	D	2084.0	1.9	1683.3	-	-	-
4	3	M	2.5	A3	2	D	2084.0	2.2	2075.3	-	-	-
3	3	M	2.5	A3	3	D	3126.0	2.8	2703.3	-	-	-
2	3	M	2.5	A3	3	D	3126.0	3.3	3087.9	-	-	-
1	3	M	2.5	A3	3	D	3126.0	3.1	2982.0	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.33	347.9	13.03	6.40	2.03
5	2	M	2.5	A3	2	S	1042.0	0.52	684.5	20.84	16.05	1.30
4	2	M	2.5	A3	2	D	2084.0	1.14	1622.2	26.05	23.68	1.10
3	2	M	2.5	A3	2	D	2084.0	1.26	1807.4	33.87	29.31	1.16
2	2	M	2.5	A3	2	D	2084.0	1.40	1966.6	33.87	32.99	1.03
1	2	M	2.5	A3	3	D	3126.0	1.74	2484.1	39.08	34.79	1.12

Table B. 50. Archetype 40 (4_3_ 6_1_HR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.3	338.7	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.5	695.1	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.6	907.7	-	-	-
3	2	M	2.5	A3	2	S	1042.0	0.6	822.7	-	-	-
2	2	M	2.5	A3	2	D	2084.0	1.0	1336.1	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.0	1403.2	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.89	611.7	13.03	6.28	2.07
5	3	M	2.5	A3	3	S	1563.0	1.58	1399.1	16.93	13.97	1.21
4	3	M	2.5	A3	2	D	2084.0	2.17	2068.0	20.84	20.05	1.04
3	3	M	2.5	A3	3	D	3126.0	2.92	2723.6	28.66	24.54	1.17
2	3	M	2.5	A3	3	D	3126.0	3.11	2772.4	33.87	27.47	1.23
1	3	M	2.5	A3	3	D	3126.0	3.15	2918.6	33.87	28.91	1.17

Table B. 51. Archetype 52 (4_3_ 6_1_MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	2	S	521.0	0.41	410.1	-	-	-
5	1	S	5	A3	3	D	1563.0	1.04	1418.2	-	-	-
4	1	S	5	A3	5	D	2605.0	1.58	2161.5	-	-	-
3	1	S	5	A3	5	D	2605.0	1.92	2556.6	-	-	-
2	1	S	5	A3	5	D	2605.0	1.87	2545.8	-	-	-
1	1	S	5	A3	6	D	3126.0	2.05	2871.5	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.88	580.0	10.42	6.40	1.63
5	3	M	2.5	A3	3	S	1563.0	1.31	1194.2	19.54	16.05	1.22
4	3	M	2.5	A3	2	D	2084.0	1.88	1715.7	28.66	23.68	1.21
3	3	M	2.5	A3	3	D	3126.0	2.48	2203.9	36.47	29.31	1.24
2	3	M	2.5	A3	3	D	3126.0	2.97	2701.8	36.47	32.99	1.11
1	3	M	2.5	A3	3	D	3126.0	2.92	2723.7	39.08	34.79	1.12

Table B. 52. Archetype 64 (4_3_ 6_1_MR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	2	S	521.0	0.4	387.9	-	-	-
5	1	S	5	A3	6	S	1563.0	1.2	1370.9	-	-	-
4	1	S	5	A3	5	D	2605.0	1.8	2206.1	-	-	-
3	1	S	5	A3	5	D	2605.0	2.0	2429.5	-	-	-
2	1	S	5	A3	6	D	3126.0	2.3	2861.9	-	-	-
1	1	S	5	A3	6	D	3126.0	2.3	2926.9	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.87	578.8	10.42	6.28	1.66
5	3	M	2.5	A3	2	S	1042.0	1.20	948.5	15.63	13.97	1.12
4	3	M	2.5	A3	3	S	1563.0	1.46	1202.4	24.75	20.05	1.23
3	3	M	2.5	A3	2	D	2084.0	2.00	1652.4	28.66	24.54	1.17
2	3	M	2.5	A3	2	D	2084.0	2.11	1755.2	31.26	27.47	1.14
1	3	M	2.5	A3	2	D	2084.0	2.20	1902.8	31.26	28.91	1.08

Table B. 53. Archetype 54 (4_B_6_1_ MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.9	563.2	-	-	-
5	3	M	2.5	A3	2	S	1042.0	1.2	917.6	-	-	-
4	3	M	2.5	A3	2	D	2084.0	1.9	1556.1	-	-	-
3	3	M	2.5	A3	2	D	2084.0	2.2	1812.3	-	-	-
2	3	M	2.5	A3	2	D	2084.0	2.3	1911.8	-	-	-
1	3	M	2.5	A3	2	D	2084.0	2.3	1913.0	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	2	S	521.0	0.33	297.4	10.42	5.71	1.82
5	1	S	5	A3	3	D	1563.0	1.26	1487.1	15.63	14.32	1.09
4	1	S	5	A3	4	D	2084.0	1.55	1890.5	26.05	21.12	1.23
3	1	S	5	A3	5	D	2605.0	2.00	2511.9	28.66	26.15	1.10
2	1	S	5	A3	6	D	3126.0	2.39	3019.3	31.26	29.44	1.06
1	1	S	5	A3	7	D	3647.0	2.67	3337.5	33.87	31.04	1.09

Table B. 54. Archetype 66 (4_B_6_1_ MR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.8	514.2	-	-	-
5	3	M	2.5	A3	2	S	1042.0	1.1	834.3	-	-	-
4	3	M	2.5	A3	2	S	1042.0	1.2	1035.6	-	-	-
3	3	M	2.5	A3	2	D	2084.0	1.9	1592.1	-	-	-
2	3	M	2.5	A3	2	D	2084.0	2.1	1717.6	-	-	-
1	3	M	2.5	A3	2	D	2084.0	2.1	1828.0	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.37	349.4	11.72	5.60	2.09
5	1	S	5	A3	5	S	1302.5	1.07	1241.1	14.33	12.46	1.15
4	1	S	5	A3	4	D	2084.0	1.58	2024.1	18.24	17.89	1.02
3	1	S	5	A3	4	D	2084.0	1.60	1990.8	26.05	21.89	1.19
2	1	S	5	A3	5	D	2605.0	1.91	2325.9	28.66	24.51	1.17
1	1	S	5	A3	5	D	2605.0	1.88	2415.8	28.66	25.79	1.11

Table B. 55. Archetype 50 (4_3_ 4_1_MR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	A3	2	S	1042.0	1.0	819.6	-	-	-
3	3	M	2.5	A3	2	D	2084.0	2.0	1830.7	-	-	-
2	3	M	2.5	A3	3	D	3126.0	2.6	2503.9	-	-	-
1	3	M	2.5	A3	3	D	3126.0	3.2	2899.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	1	S	5	A3	5	S	1302.5	0.94	1164.5	14.33	11.97	1.20
3	1	S	5	A3	6	D	3126.0	2.10	2925.8	31.26	28.36	1.10
2	1	S	5	A3	8	D	4168.0	2.89	4101.5	44.29	39.29	1.13
1	1	S	5	A3	9	D	4689.0	3.34	4611.8	46.89	44.81	1.05

Table B. 56. Archetype 62 (4_3_ 4_1_MR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	A3	2	S	1042.0	1.0	773.9	-	-	-
3	3	M	2.5	A3	3	S	1563.0	1.3	1333.5	-	-	-
2	3	M	2.5	A3	3	D	3126.0	2.3	2145.3	-	-	-
1	3	M	2.5	A3	3	D	3126.0	2.9	3036.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	1	S	5	A3	5	S	1302.5	0.99	1174.8	14.33	11.68	1.23
3	1	S	5	A3	6	D	3126.0	1.94	2932.8	27.35	24.67	1.11
2	1	S	5	A3	7	D	3647.0	2.44	3446.6	41.68	33.32	1.25
1	1	S	5	A3	7	D	3647.0	1.92	2986.0	41.68	37.71	1.11

Table B. 57. Archetype 76 (4_E_6_2_ LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	5	S	5	A3	3	S	781.5	0.47	403.1	19.54	10.08	1.94
5	5	S	5	A3	4	S	1042.0	0.96	1010.6	26.05	25.27	1.03
4	5	S	5	A3	3	D	1563.0	1.29	1491.0	39.08	37.28	1.05
3	5	S	5	A3	4	D	2084.0	1.68	1846.0	52.10	46.15	1.13
2	5	S	5	A3	4	D	2084.0	1.74	2077.8	52.10	51.94	1.00
1	5	S	5	A3	5	D	2605.0	1.79	2190.7	65.13	54.77	1.19

Table B. 58. Archetype 88 (4_E_6_2_ LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	5	S	5	A3	2	S	521.0	0.42	395.5	13.03	9.89	1.32
5	5	S	5	A3	4	S	1042.0	0.83	879.7	26.05	21.99	1.18
4	5	S	5	A3	5	S	1302.5	1.07	1262.6	32.56	31.57	1.03
3	5	S	5	A3	3	D	1563.0	1.29	1545.5	39.08	38.64	1.01
2	5	S	5	A3	4	D	2084.0	1.40	1730.2	52.10	43.26	1.20
1	5	S	5	A3	4	D	2084.0	1.54	1820.4	52.10	45.51	1.14

Table B. 59. Archetype 100 (4_E_6_2_ HR_HG_ DX_LP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.37	403.1	5.21	2.02	2.59
5	2	M	2.5	A3	2	S	1042.0	0.67	1009.6	5.21	5.05	1.03
4	2	M	2.5	A3	2	D	2084.0	1.06	1493.2	10.42	7.46	1.40
3	2	M	2.5	A3	3	D	3126.0	1.34	1845.5	15.63	9.23	1.69
2	2	M	2.5	A3	3	D	3126.0	1.42	2080.0	15.63	10.39	1.50
1	2	M	2.5	A3	3	D	3126.0	1.52	2192.1	15.63	10.95	1.43

* There are five of (2)2.5ft walls along the wall line and all take equal load.

Table B. 60. Archetype 112 (4_E_6_2_ HR_LG_ DX_LP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.37	395.5	5.21	1.98	2.63
5	2	M	2.5	A3	2	S	1042.0	0.62	879.7	5.21	4.40	1.18
4	2	M	2.5	A3	2	S	1563.0	0.88	1262.6	7.82	6.31	1.24
3	2	M	2.5	A3	2	D	2084.0	1.07	1545.5	10.42	7.73	1.35
2	2	M	2.5	A3	2	D	2084.0	1.21	1730.0	10.42	8.65	1.20
1	2	M	2.5	A3	2	D	2084.0	1.27	1820.7	10.42	9.10	1.14

* There are five of (2)2.5ft walls along the wall line and all take equal load.

Table B. 61. Archetype 124 (4_E_6_2_ MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A	2	S	1042.0	0.30	291.0	-	-	-
5	2	M	2.5	A	2	S	1042.0	0.46	518.5	-	-	-
4	2	M	2.5	A	2	S	1042.0	0.53	647.8	-	-	-
3	2	M	2.5	A	2	S	1042.0	0.54	634.5	-	-	-
2	2	M	2.5	A	2	S	1042.0	0.60	757.1	-	-	-
1	2	M	2.5	A	2	D	2084.0	1.02	1268.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A	2	S	1042.0	0.30	291.0	-	-	-
5	2	M	2.5	A	2	S	1042.0	0.46	518.5	-	-	-
4	2	M	2.5	A	2	S	1042.0	0.53	647.8	-	-	-
3	2	M	2.5	A	2	S	1042.0	0.54	634.5	-	-	-
2	2	M	2.5	A	2	S	1042.0	0.60	750.4	-	-	-
1	2	M	2.5	A	2	D	2084.0	0.95	1181.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	S	5	A	3	S	781.5	0.50	477.8	22.14	10.08	2.20
5	3	S	5	A	6	S	1563.0	1.18	1338.7	33.87	25.27	1.34
4	3	S	5	A	4	D	2084.0	1.69	2053.2	41.68	37.28	1.12
3	3	S	5	A	6	D	3126.0	2.24	2653.7	57.31	46.15	1.24
2	3	S	5	A	6	D	3126.0	2.36	2960.5	57.31	51.94	1.10
1	3	S	5	A	6	D	3126.0	2.28	2834.6	67.73	54.77	1.24

Table B. 62. Archetype 136 (4_E_6_2_ MR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A	2	S	1042.0	0.30	284.8	-	-	-
5	2	M	2.5	A	2	S	1042.0	0.43	472.1	-	-	-
4	2	M	2.5	A	2	S	1042.0	0.48	545.1	-	-	-
3	2	M	2.5	A	2	S	1042.0	0.53	612.1	-	-	-
2	2	M	2.5	A	2	S	1042.0	0.55	647.8	-	-	-
1	2	M	2.5	A	2	S	1042.0	0.59	733.5	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A	2	S	1042.0	0.30	284.8	-	-	-
5	2	M	2.5	A	2	S	1042.0	0.43	472.1	-	-	-
4	2	M	2.5	A	2	S	1042.0	0.48	545.1	-	-	-
3	2	M	2.5	A	2	S	1042.0	0.53	612.1	-	-	-
2	2	M	2.5	A	2	S	1042.0	0.55	647.8	-	-	-
1	2	M	2.5	A	2	S	1042.0	0.55	683.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	S	5	A	3	S	781.5	0.49	469.4	22.14	9.89	2.24
5	3	S	5	A	5	S	1302.5	1.05	1151.4	29.96	21.99	1.36
4	3	S	5	A	4	D	2084.0	1.53	1740.9	41.68	31.57	1.32
3	3	S	5	A	5	D	2605.0	1.86	2167.8	49.50	38.64	1.28
2	3	S	5	A	5	D	2605.0	2.09	2451.8	49.50	43.26	1.14
1	3	S	5	A	5	D	2605.0	2.06	2561.6	49.50	45.51	1.09

INPUT DATA					DESIGN SUMMARY		
Total Height	h_n=	60.0	ft		Total base shear		
Total Weight	W=	1,055	k		V	=	262.14
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					

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 Bldg. 4, 6 story, R=4

INPUT DATA					DESIGN SUMMARY		
Total Height	h_n=	60.0	ft		Total base shear		
Total Weight	W=	1,270	k		V	=	315.47
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					

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 Bldg. 4, 6 story, R=4

Table B. 63. Tributary load calculation, Index Bldg. 4, 6 story, N-S direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: 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 B. 64. Tributary load calculation, Index Bldg. 4, 6 story, E-W direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 65. Archetype 28 (4_3_ 6_1_HR_HG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.25	227.7	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.48	655.6	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.60	814.9	-	-	-
3	2	M	2.5	A3	2	D	2084.0	1.00	1406.2	-	-	-
2	2	M	2.5	A3	3	S	1563.0	0.80	1114.1	-	-	-
1	2	M	2.5	A3	3	S	1563.0	0.69	982.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.80	227.7	13.03	4.80	2.71
5	3	M	2.5	A3	3	S	1563.0	1.29	655.6	16.93	12.04	1.41
4	3	M	2.5	A3	2	D	2084.0	2.01	814.9	20.84	17.76	1.17
3	3	M	2.5	A3	2	D	2084.0	2.12	1406.2	26.05	21.98	1.18
2	3	M	2.5	A3	3	D	3126.0	2.75	1114.1	31.26	24.74	1.26
1	3	M	2.5	A3	3	D	3126.0	2.99	982.7	31.26	26.09	1.20

Table B. 66. Archetype 40 (4_3_ 6_1_HR_LG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.24	220.9	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.49	605.1	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.58	793.6	-	-	-
3	2	M	2.5	A3	2	S	1042.0	0.54	767.6	-	-	-
2	2	M	2.5	A3	2	S	1042.0	0.60	839.5	-	-	-
1	2	M	2.5	A3	2	S	1042.0	0.45	609.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	2.5	A3	2	S	1042.0	0.79	480.8	13.03	4.71	2.77
5	3	M	2.5	A3	2	S	1042.0	1.21	993.5	13.03	10.48	1.24
4	3	M	2.5	A3	3	S	1563.0	1.62	1475.7	16.93	15.04	1.13
3	3	M	2.5	A3	2	D	2084.0	2.06	1942.3	20.84	18.41	1.13
2	3	M	2.5	A3	3	D	3126.0	2.33	2187.7	28.66	20.60	1.39
1	3	M	2.5	A3	3	D	3126.0	2.76	2484.1	28.66	21.68	1.32

Table B. 67. Archetype 100 (4_E_6_2_ HR_HG_ DX_LP) design*, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.31	302.3	5.21	1.51	3.45
5	2	M	2.5	A3	2	S	1042.0	0.61	757.9	5.21	3.79	1.37
4	2	M	2.5	A3	3	S	1563.0	0.85	1118.5	7.82	5.59	1.40
3	2	M	2.5	A3	3	S	1563.0	0.99	1384.5	7.82	6.92	1.13
2	2	M	2.5	A3	3	S	1563.0	1.06	1558.4	7.82	7.79	1.00
1	2	M	2.5	A3	2	D	2084.0	1.21	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.

Table B. 68. Archetype 112 (4_E_6_2_ HR_LG_ DX_LP) design*, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.31	296.7	5.21	1.48	3.51
5	2	M	2.5	A3	2	S	1042.0	0.54	659.8	5.21	3.30	1.58
4	2	M	2.5	A3	2	S	1042.0	0.69	947.0	5.21	4.73	1.10
3	2	M	2.5	A3	3	S	1563.0	0.87	1159.1	7.82	5.80	1.35
2	2	M	2.5	A3	3	S	1563.0	0.97	1297.6	7.82	6.49	1.20
1	2	M	2.5	A3	3	S	1563.0	1.02	1365.4	7.82	6.83	1.14

* There are five of (2)2.5ft walls along the wall line and all take equal load.

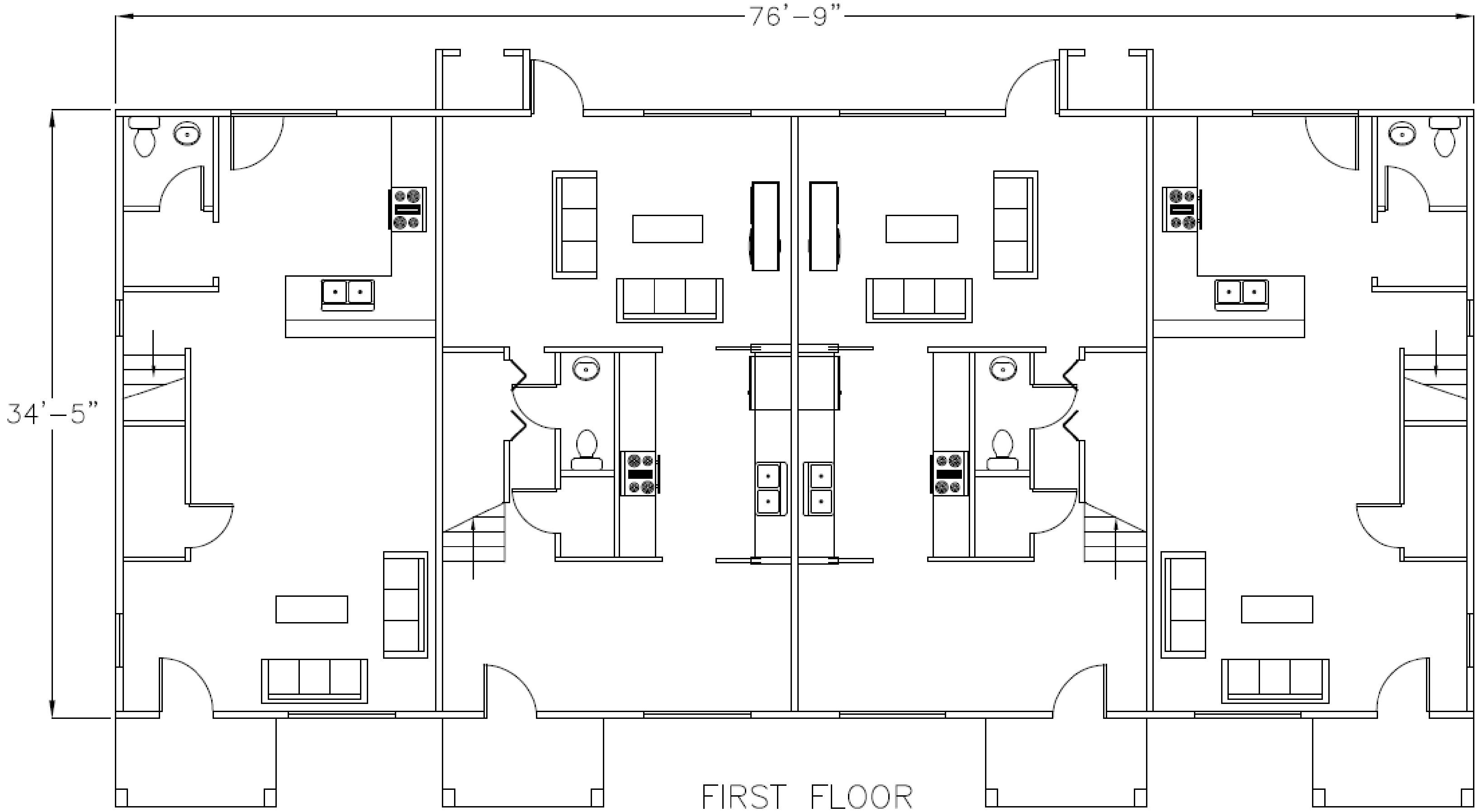


Figure B. 29. Index Bldg. 5 first floor plan

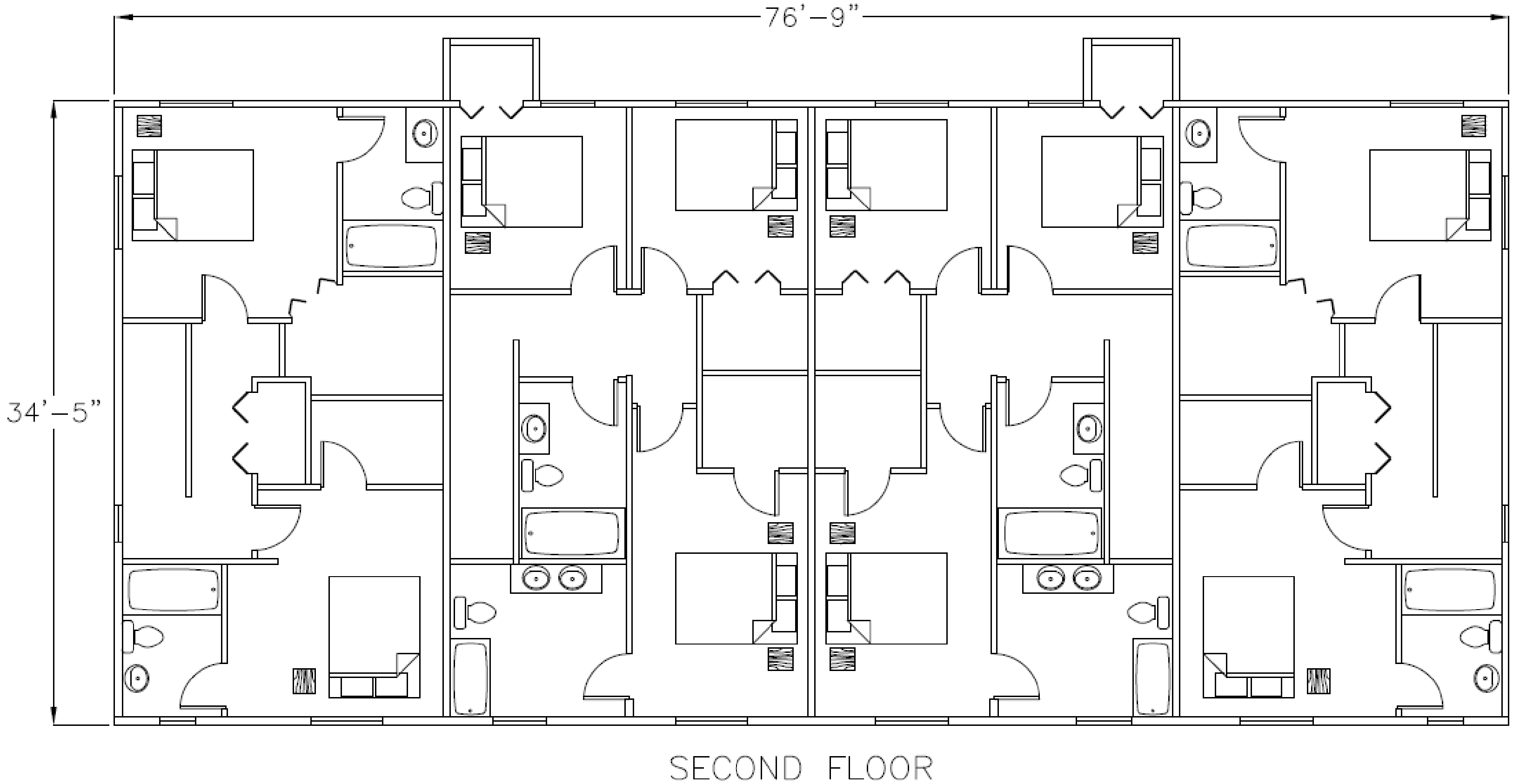


Figure B. 30. Index Bldg. 5 second floor plan

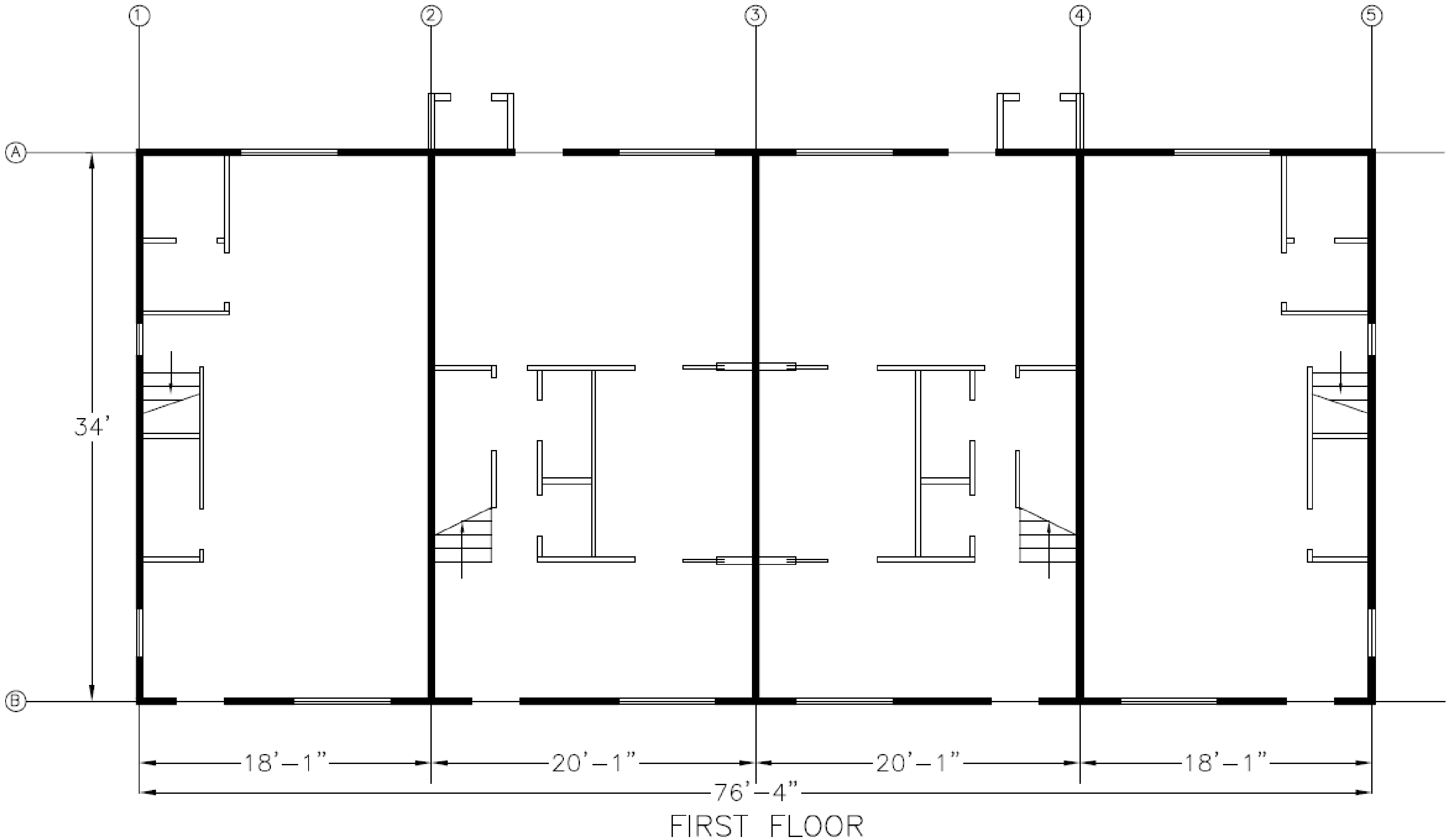


Figure B. 31. Index Bldg. 5 first floor assigned shear walls

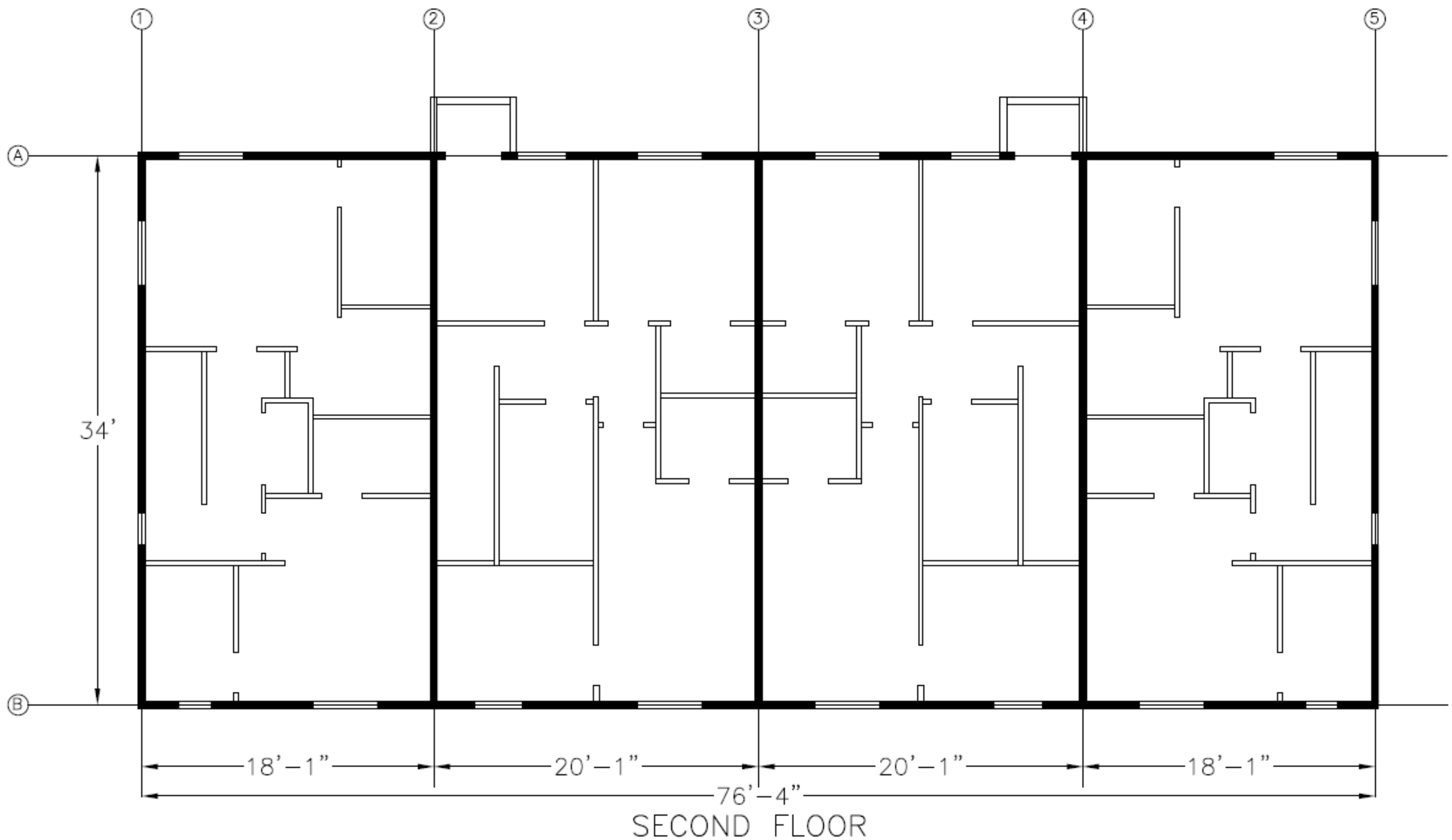


Figure B. 32. Index Bldg. 5 second floor assigned shear walls

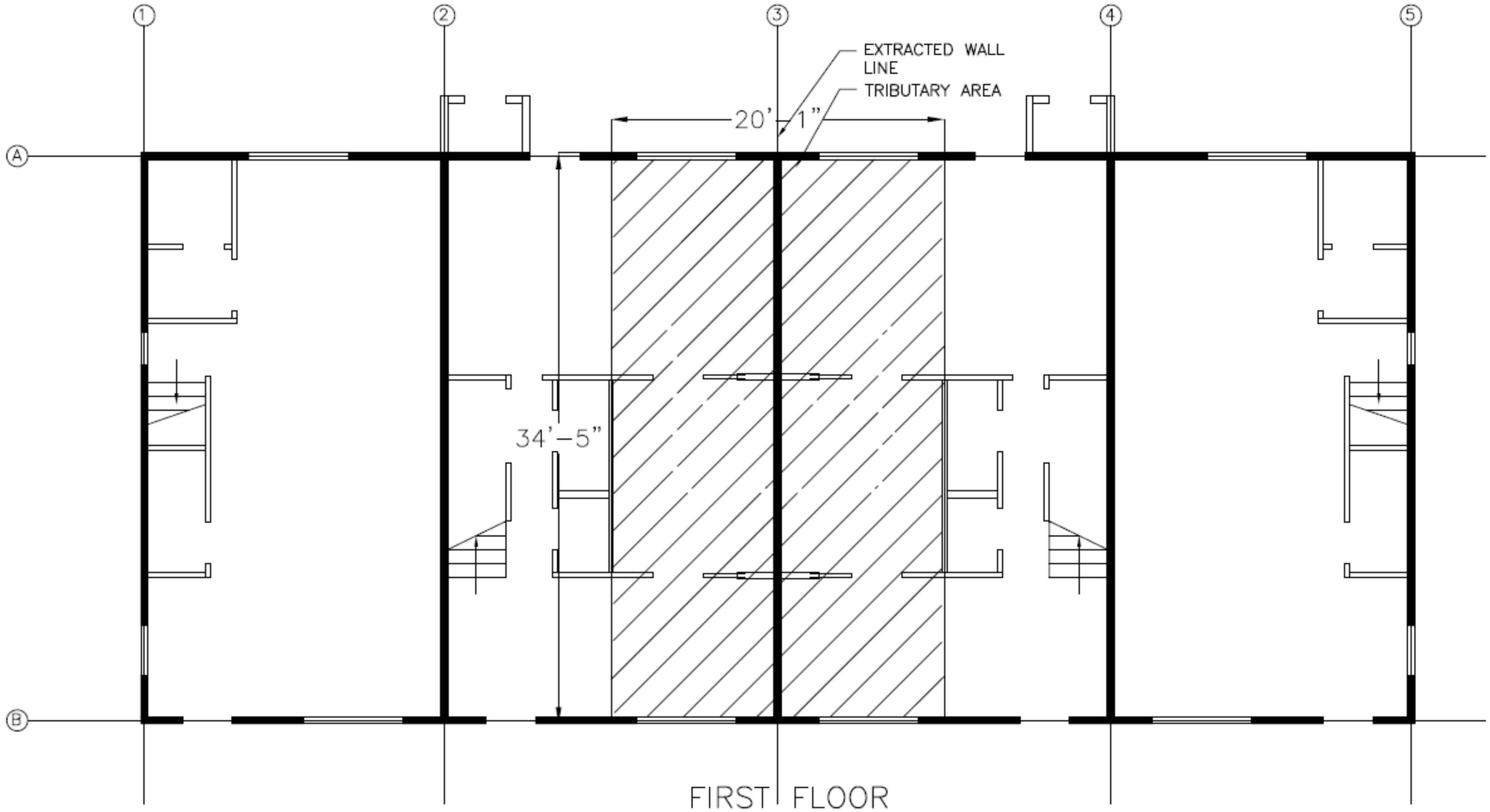


Figure B. 33. Index Bldg. 5 first floor tributary area

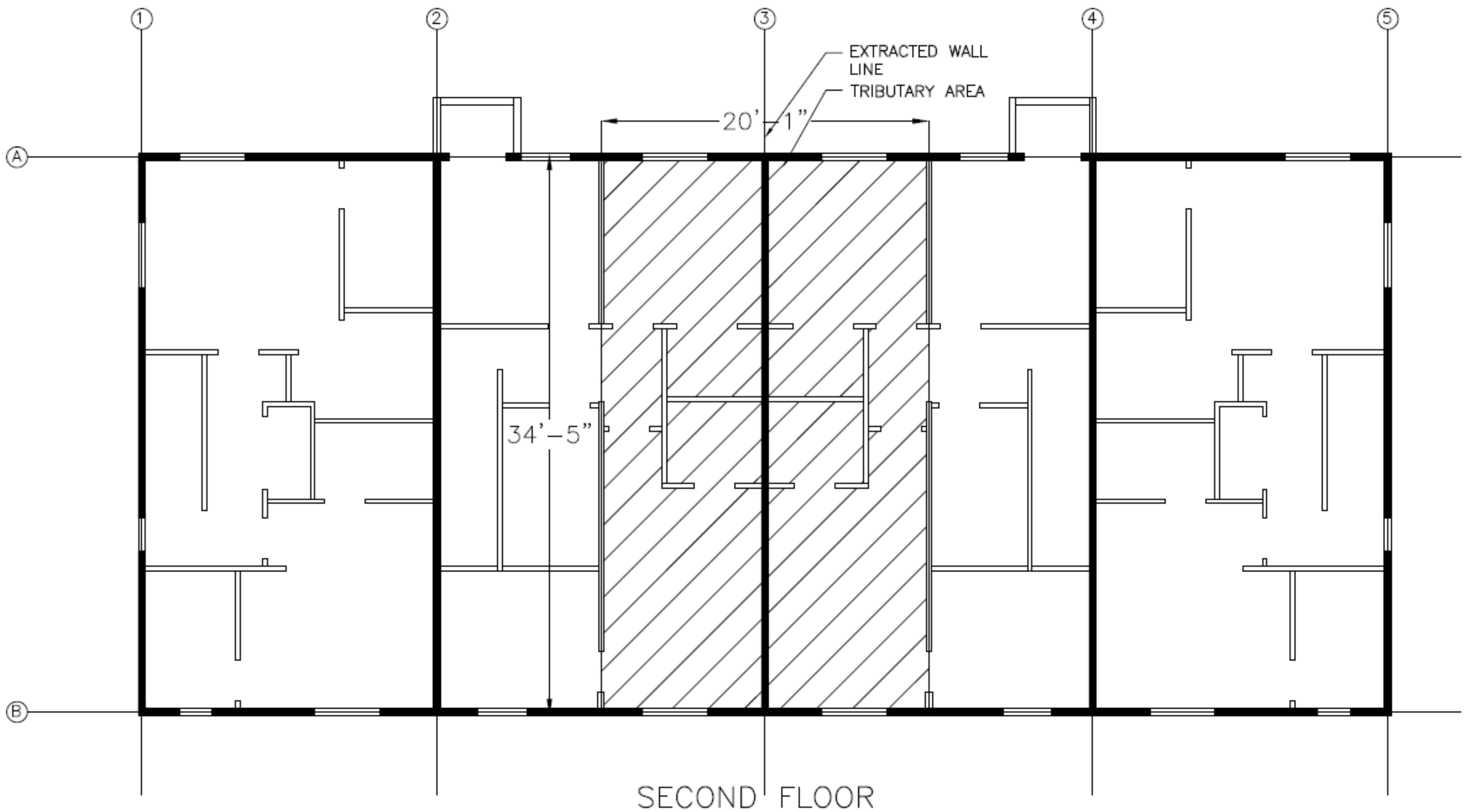


Figure B. 34. Index Bldg. 5 second floor tributary area

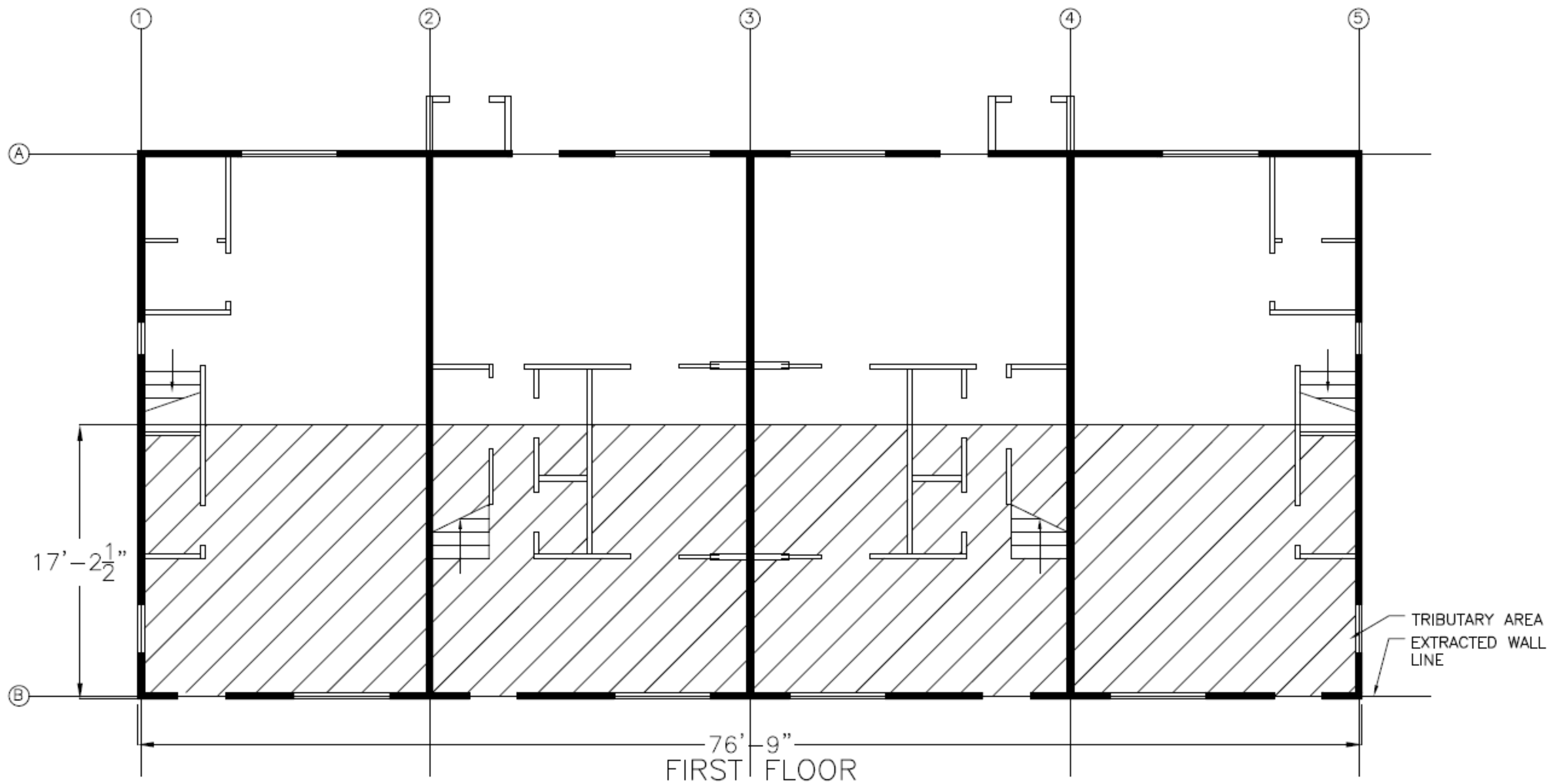


Figure B. 35. Index Bldg. 5 first floor tributary area

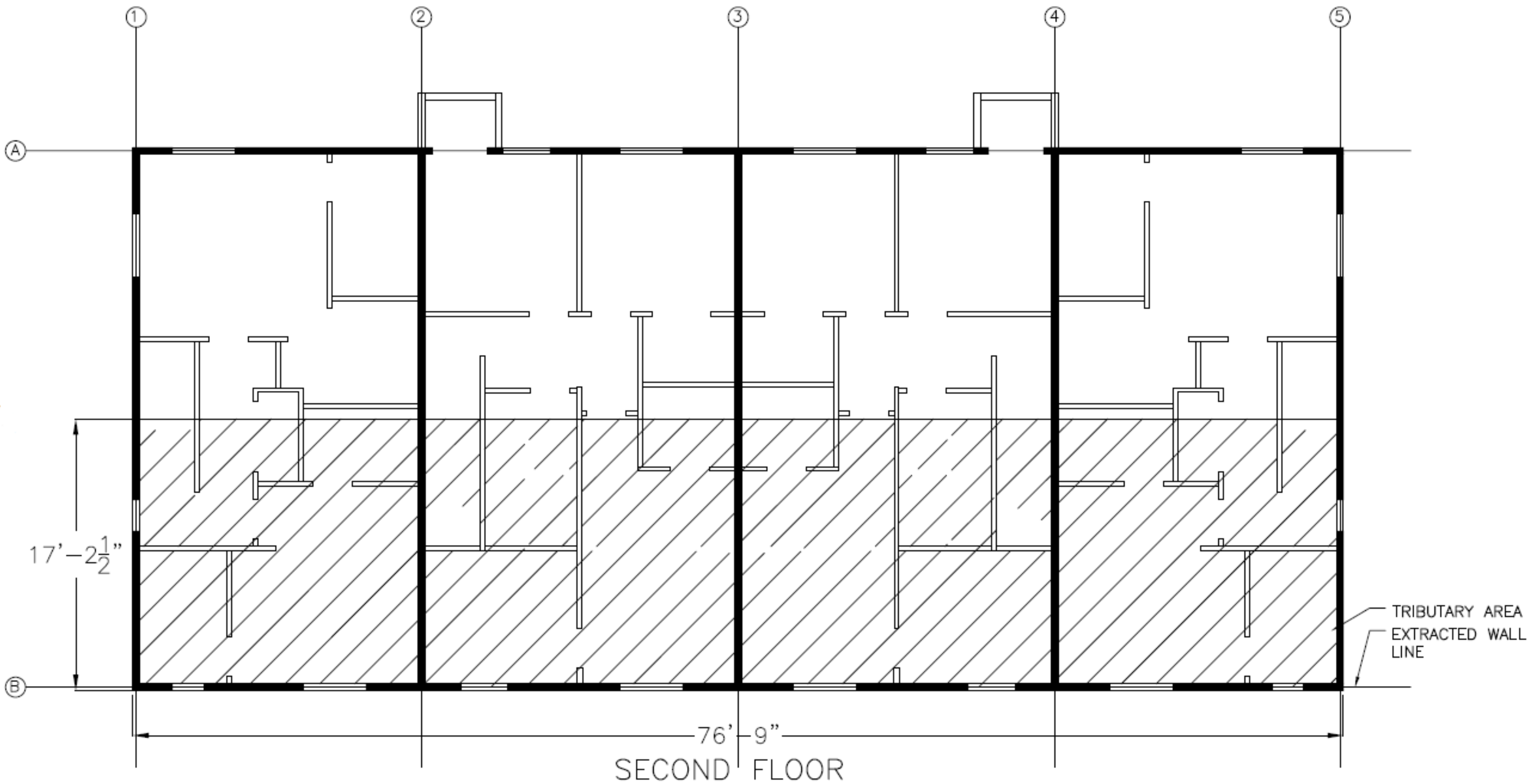


Figure B. 36. Index Bldg. 5 second floor tributary area

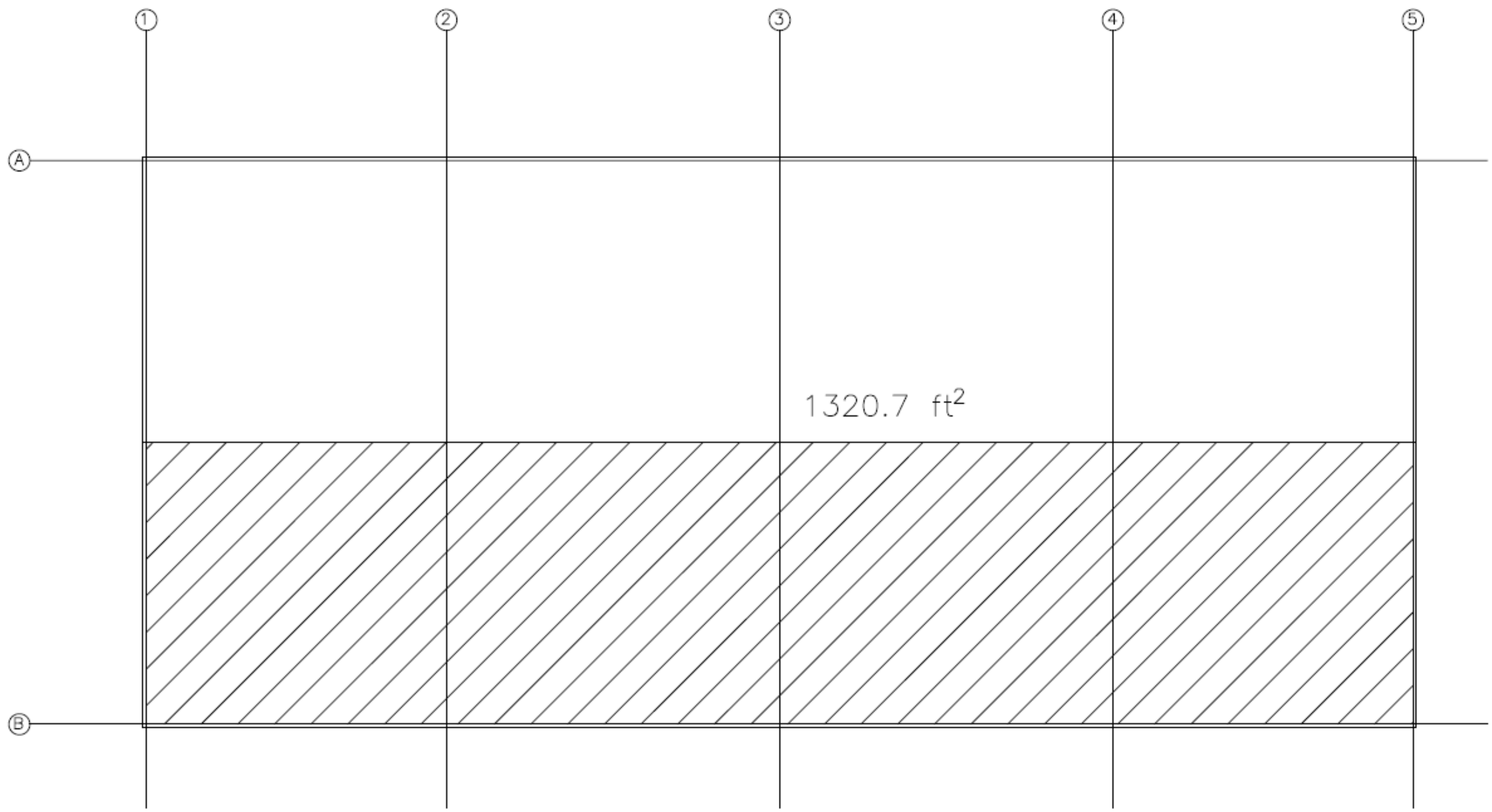


Figure B. 37. Index Bldg. 5 tributary area

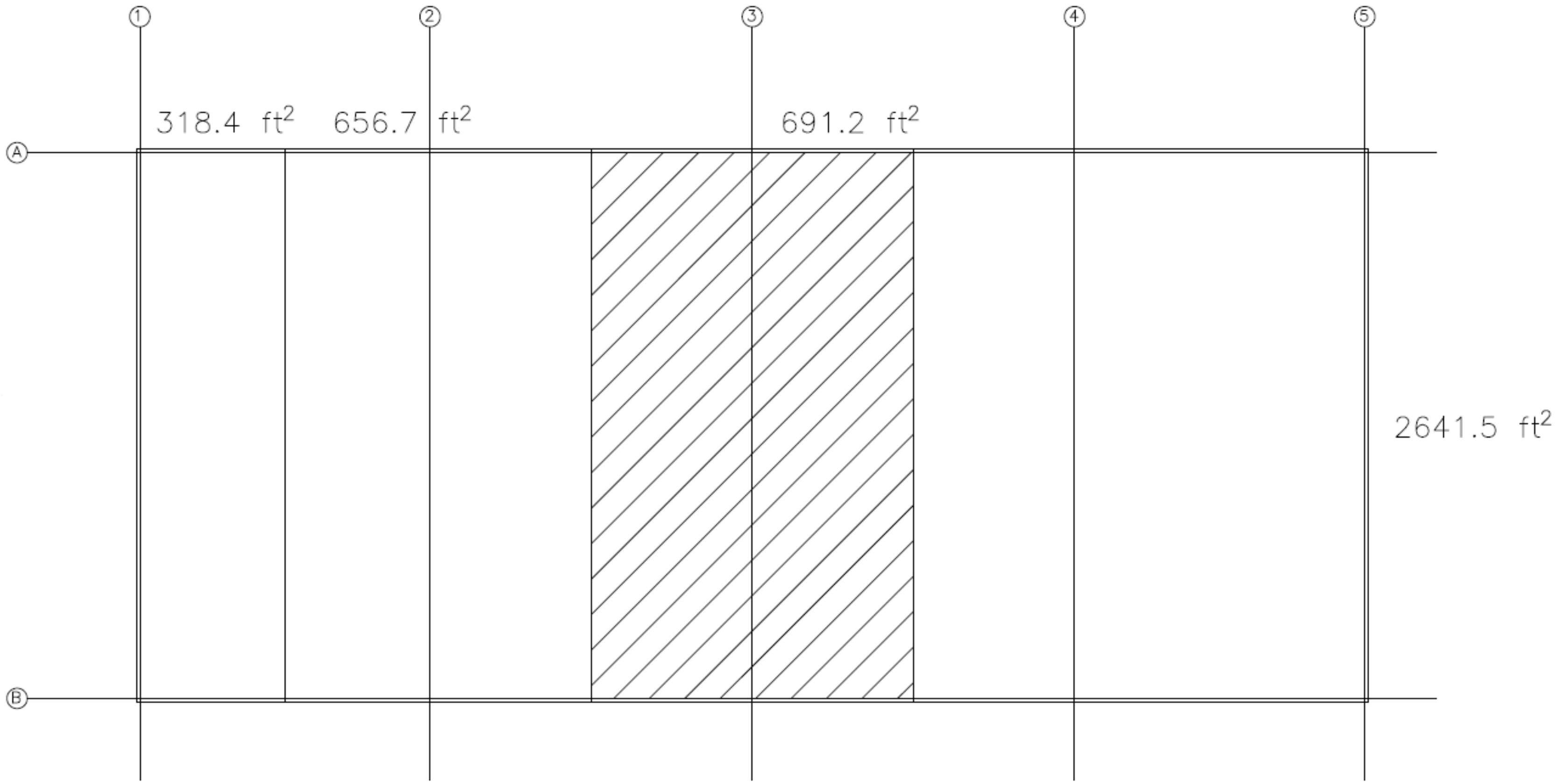
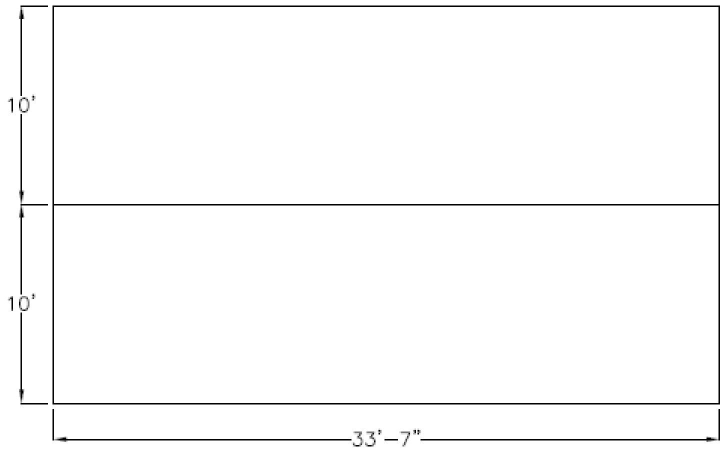
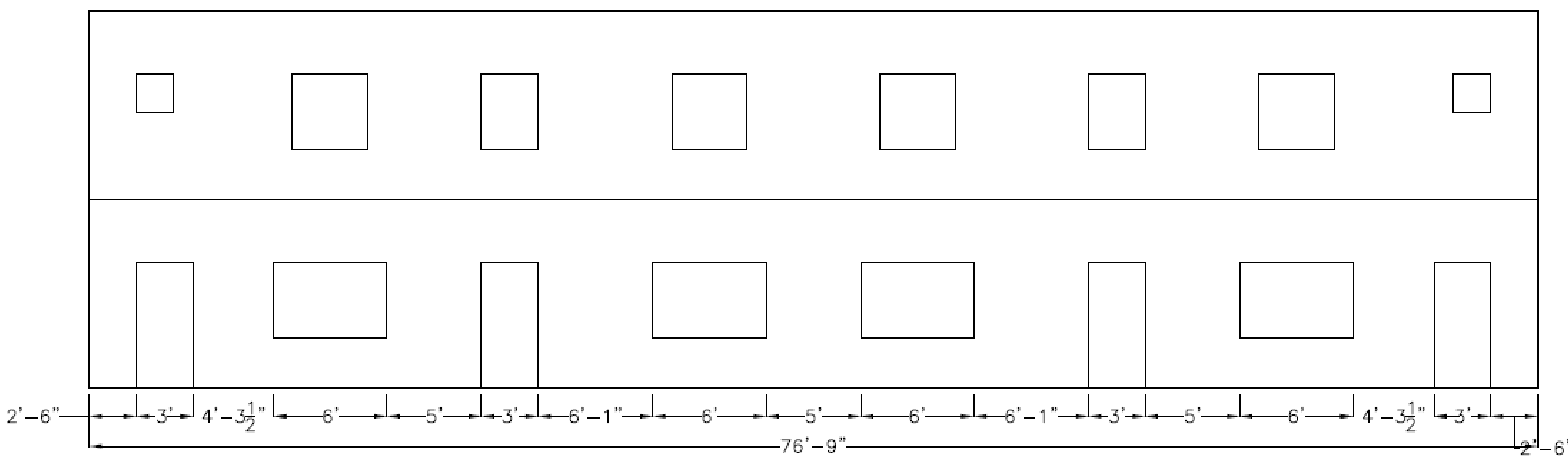


Figure B. 38. Index Bldg. 5 second floor tributary area



5_3_2



5_B_2

Figure B. 39. Index Bldg. 5 extracted shear wall lines

Table B. 69. Extracted Archetypes, Index Bldg. 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 B. 70. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 71. Seismic weight summary, Index Bldg. 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				DESIGN SUMMARY
Total Height	h_n=	20.0	ft	Total base shear
Total Weight	W=	323	k	V = 107.57
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		

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}	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 Bldg. 5, 2 story, R=3

INPUT DATA				DESIGN SUMMARY
Total Height	h_n=	20.0	ft	Total base shear
Total Weight	W=	376	k	V = 125.47
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		

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}	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 Bldg. 5, 2 story, R=3

Table B. 72. Tributary load calculation, Index Bldg. 5, 2 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)	High gravity Story shear (kip)	High gravity 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 B. 73. Archetype 75 (5_B_2_2_ LR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	4	D	2084.0	1.41	1629.7	-	-	-
1	2	S	5	A3	7	D	3647.0	2.55	3136.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	4	D	2084.0	1.41	1629.7	41.68	32.59	1.28
1	2	S	5	A3	7	D	3647.0	2.55	3136.7	72.94	62.73	1.16

Table B. 74. Archetype 87 (5_B_2_2_ LR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	3	D	1563.0	1.23	1562.5	-	-	-
1	2	S	5	A3	6	D	3126.0	2.19	2685.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	3	D	1563.0	1.23	1561.9	31.26	31.24	1.00
1	2	S	5	A3	6	D	3126.0	2.19	2692.5	62.52	53.78	1.16

Table B. 75. Archetype 123 (5_B_2_2_ MR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.45	655.7	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.09	1955.6	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.45	655.7	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.09	1955.6	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	5	D	2605.0	1.80	2603.8	36.47	32.59	1.12
1	2	S	5	A3	9	D	4689.0	2.40	4317.7	67.73	62.73	1.08

Table B. 76. Archetype 135 (5_B_2_2_ MR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.54	808.0	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.10	1814.5	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.54	808.0	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.10	1814.5	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	S	5	A3	5	D	2605.0	1.55	2316.4	36.47	31.24	1.17
1	2	S	5	A3	7	D	3647.0	2.16	3563.9	57.31	53.78	1.07

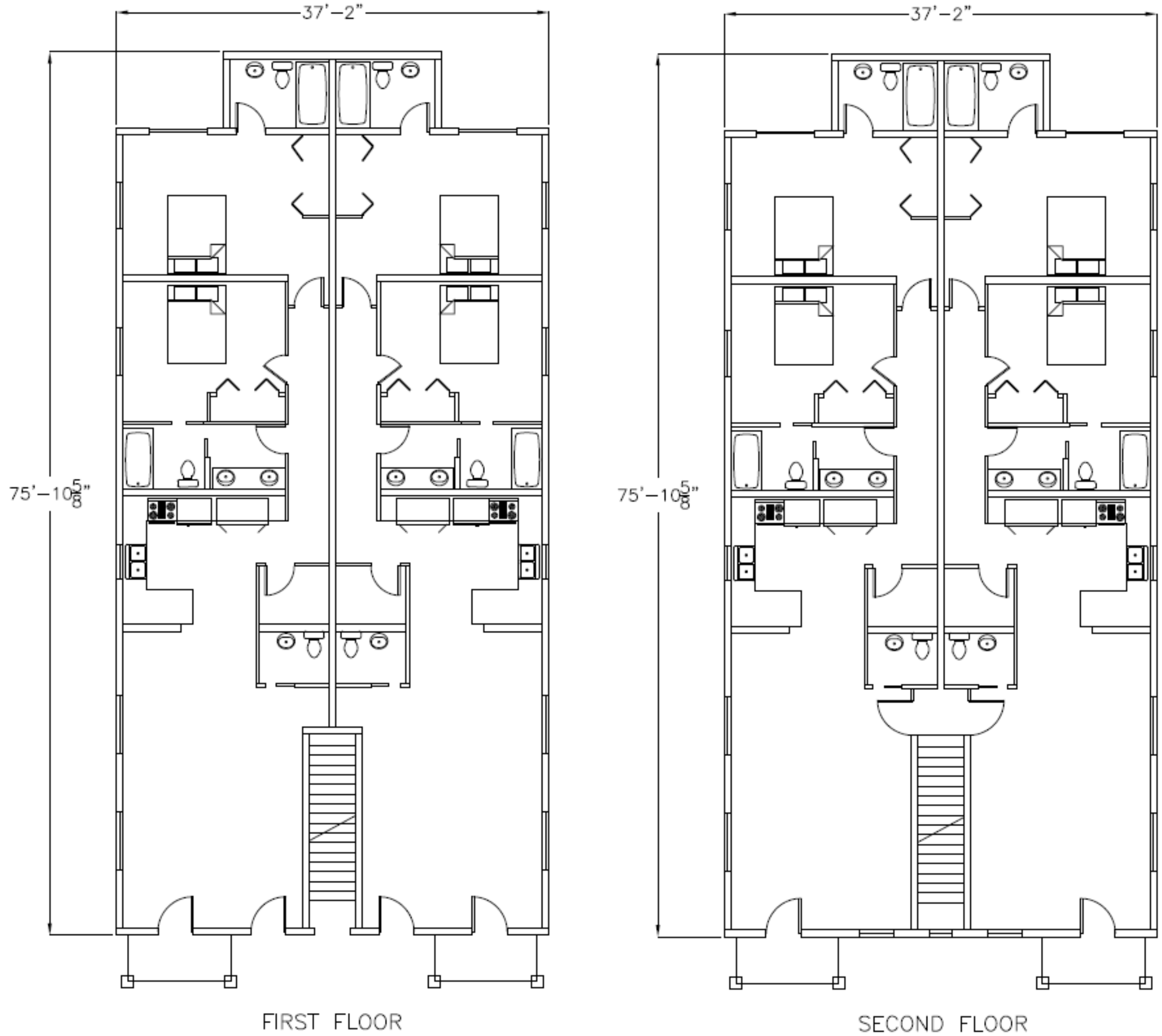


Figure B. 40. Index Bldg. 6 floor plan

Figure B. 41. Index Bldg. 6 assigned shear walls

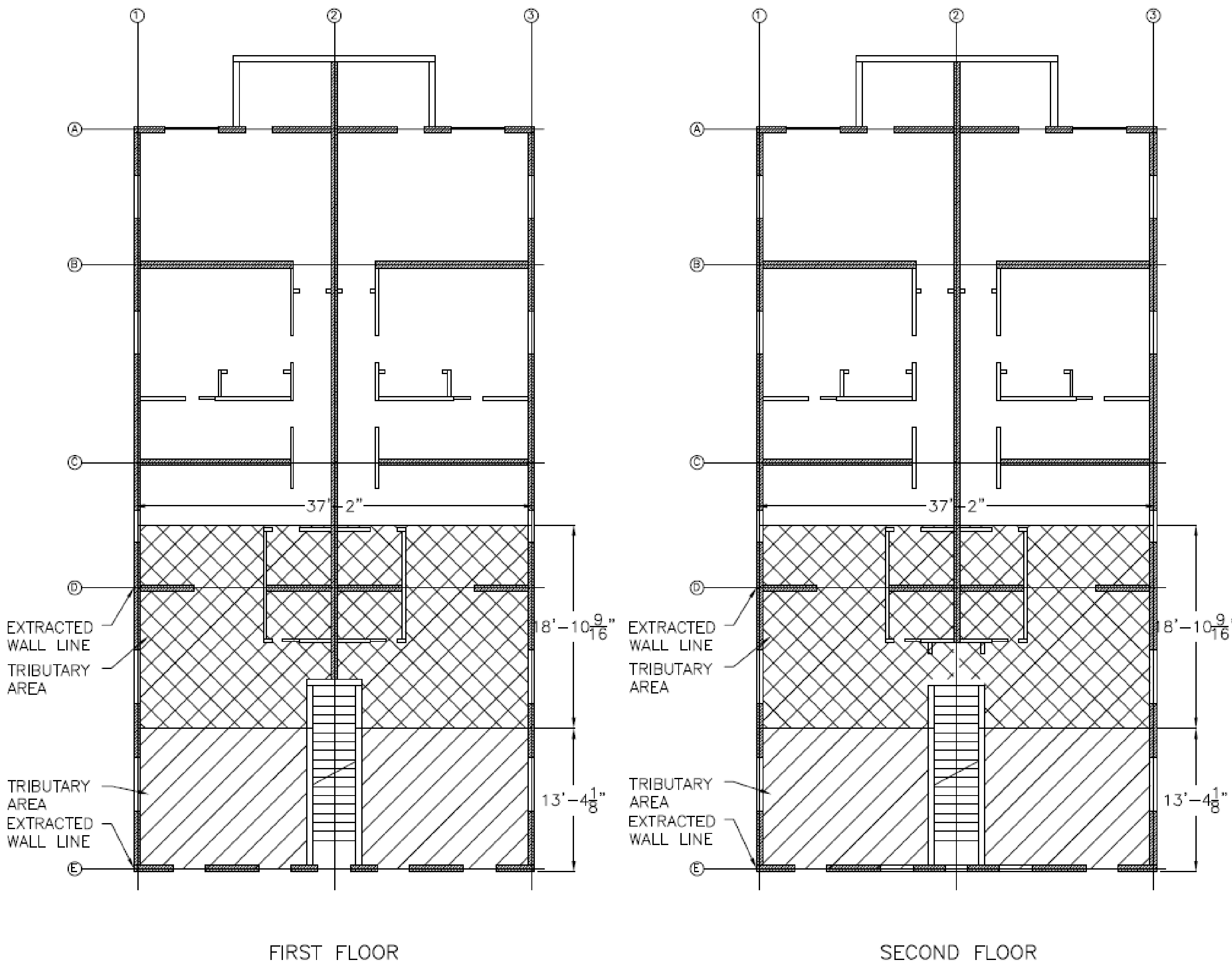


Figure B. 42. Index Bldg. 6 tributary area

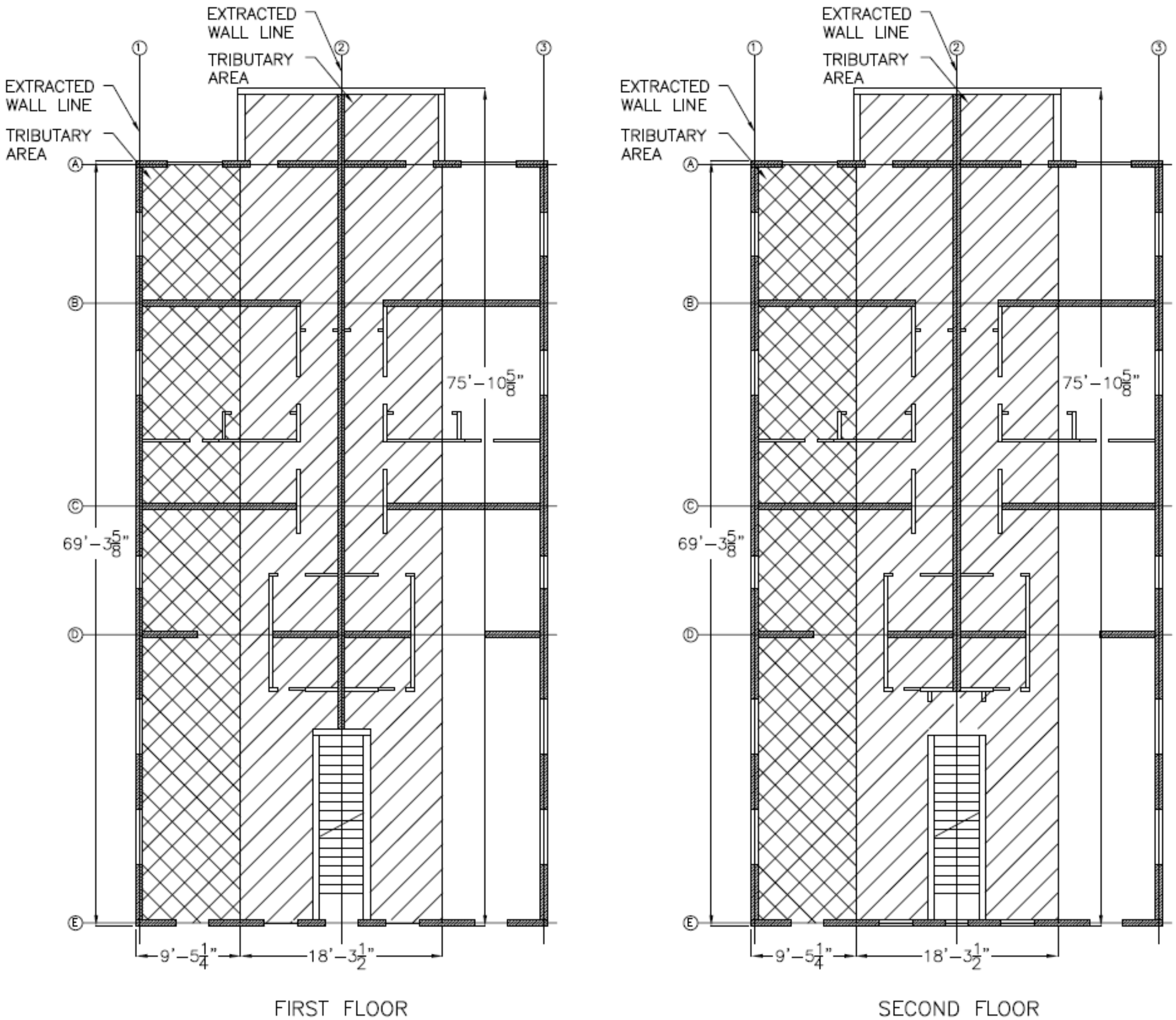


Figure B. 43. Index Bldg. 6 tributary area

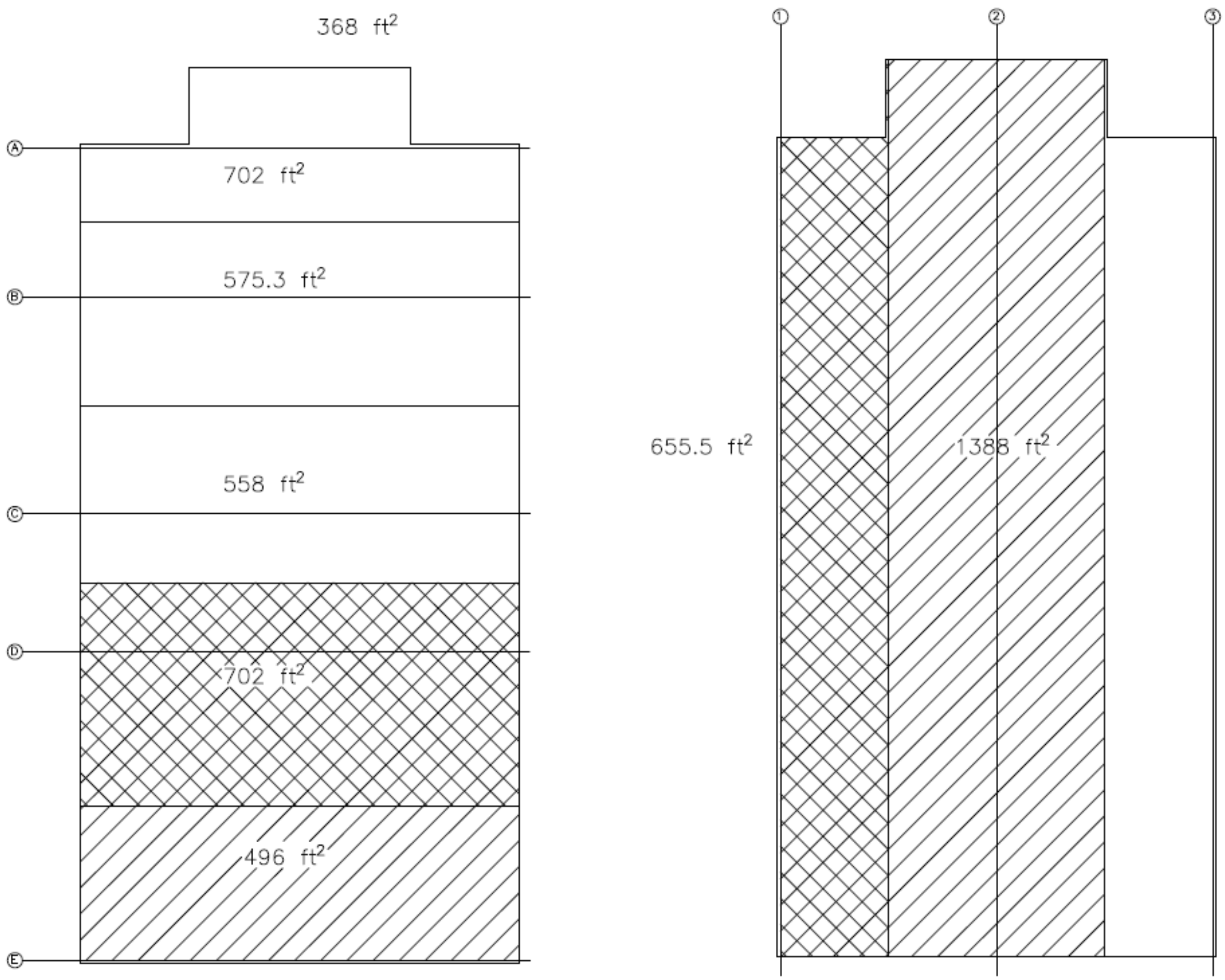
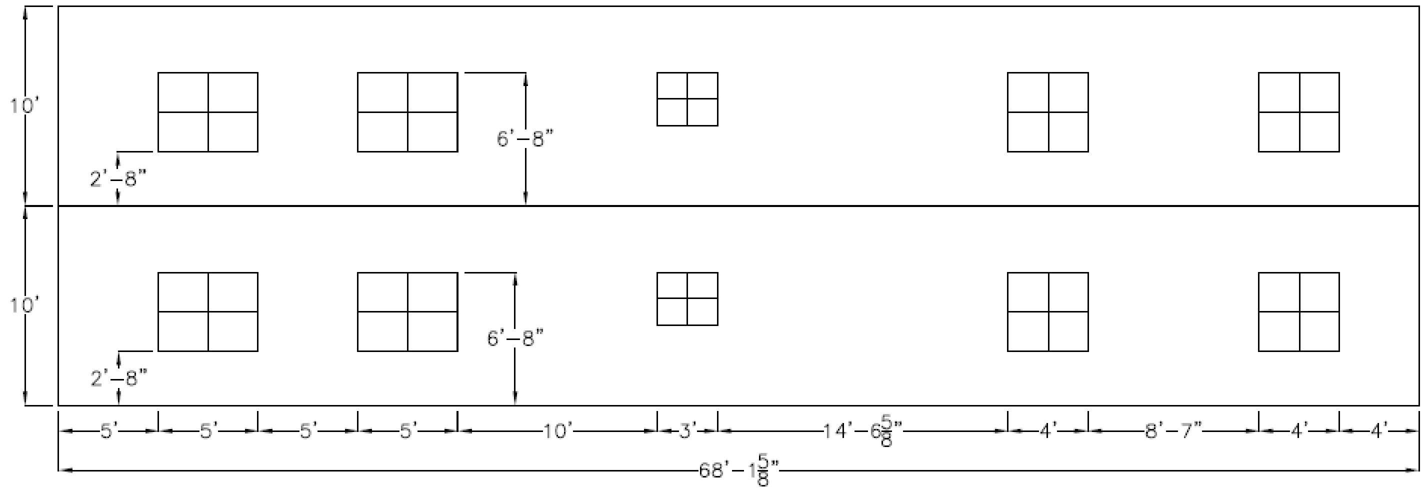


Figure B. 44. Index Bldg. 6 tributary area



6_1_2

6_2_2

Figure B. 45. Index Bldg. 6 extracted shear wall lines

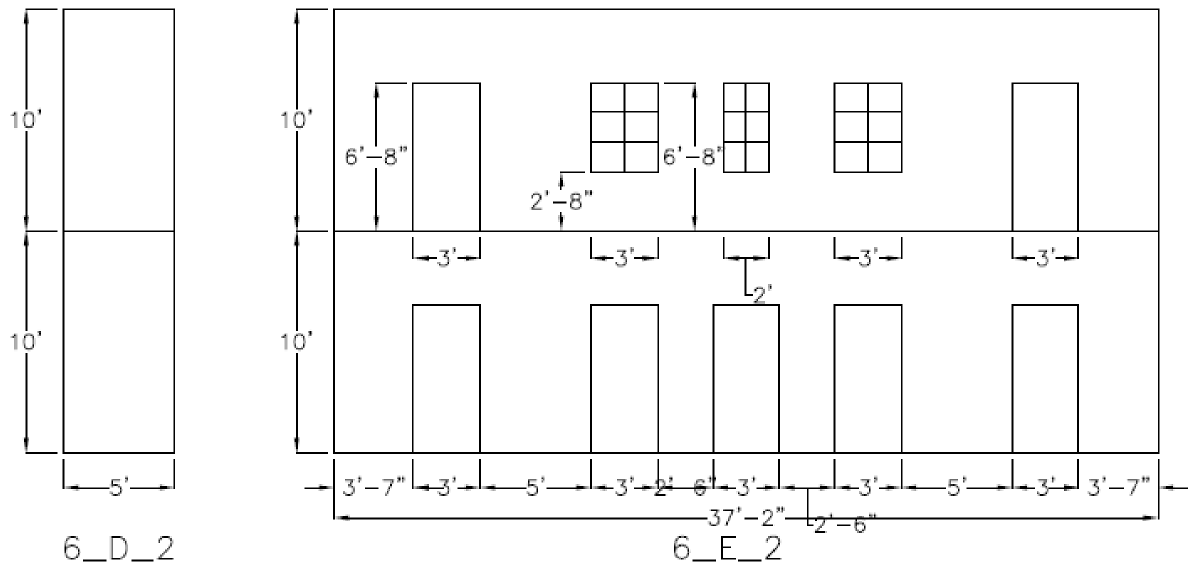


Figure B. 46. Index Bldg. 6 extracted shear wall lines

Table B. 77. Extracted Archetypes, Index Bldg. 6

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}	6_D_2_1_ LR_HG_DX_SP	3
PG-5			Low		6_D_2_1_ LR_LG_DX_SP	15
PG-9		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 B. 78. Seismic weight detailed calculation, Index Bldg. 6, 2 story

								Low gravity				High gravity			
Level	Story	h (ft)	A_{floor}	L_{xtwall}	$A_{extWall}$-Openings	$L_{Intwall}$	$A_{IntWall}$-Openings	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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	1985.333333	169.50	1695		66947	43957			66947	43957	
Ground			0												

Table B. 79. Seismic weight summary, Index Bldg. 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				DESIGN SUMMARY
Total Height	h_n=	20.0	ft	Total base shear
Total Weight	W=	330	k	V = 109.90
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		

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}	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 Bldg. 6, 2 story, R=3

INPUT DATA				DESIGN SUMMARY
Total Height	h_n=	20.0	ft	Total base shear
Total Weight	W=	379	k	V = 126.20
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		

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}	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 Bldg. 6, 2 story, R=3

Table B. 80. Tributary load calculation, Index Bldg. 6, 2 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)	High gravity Story shear (kip)	High gravity 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 B. 81. Archetype 3 (6_D_2_1_ LR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.74	858.1	5.21	4.29	1.21
1	1	S	5	A3	4	D	2084.0	1.31	1638.1	10.42	8.19	1.27

Table B. 82. Archetype 15 (6_D_2_1_ LR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	1	S	5	A3	4	S	1042.0	0.72	828.0	5.21	4.14	1.26
1	1	S	5	A3	6	S	1563.0	1.12	1428.8	7.82	7.14	1.09

Table B. 83. Archetype 27 (6_D_2_1_HR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.52	858.1	5.21	4.29	1.21
1	2	M	2.5	A3	2	D	2084.0	0.97	1638.1	10.42	8.19	1.27

Table B. 84. Archetype 39 (6_D_2_1_ HR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	S	1042.0	0.51	828.0	5.21	4.14	1.26
1	2	M	2.5	A3	2	D	2084.0	0.91	1428.8	10.42	7.14	1.46

Table B. 85. Archetype 98 (6_E_2_2_ HR_HG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	D	2084.0	0.81	1245.4	-	-	-
1	2	M	2.5	A3	3	D	3126.0	1.28	2315.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	D	2084.0	0.76	1179.7	20.84	12.13	1.72
1	2	M	2.5	A3	3	D	3126.0	1.28	2314.4	31.26	23.15	1.35

Table B. 86. Archetype 110 (6_E_2_2_ HR_LG_ DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	D	2084.0	0.79	1176.5	-	-	-
1	2	M	2.5	A3	2	D	2084.0	1.10	2014.5	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A3	2	D	2084.0	0.78	1163.6	20.84	11.70	1.78
1	2	M	2.5	A3	2	D	2084.0	1.11	2023.6	20.84	20.19	1.03

INPUT DATA				DESIGN SUMMARY	
Total Height	h_n=	20.0	ft	Total base shear	
Total Weight	W=	379	k	V =	94.65
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			

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 Bldg. 6, 2 story, R=4

INPUT DATA				DESIGN SUMMARY	
Total Height	h_n=	20.0	ft	Total base shear	
Total Weight	W=	330	k	V =	82.43
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			

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 Bldg. 6, 2 story, R=4

Table B. 87. Tributary load calculation, Index Bldg. 6, 2 story, E-W direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity		High gravity	
				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 B. 88. Archetype 27 (6_D_2_1_HR_HG_DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A	2	S	1042.0	0.46	645.1	5.21	3.23	1.62
1	2	M	2.5	A	3	S	1563.0	0.78	1230.5	7.82	6.15	1.27

Table B. 89. Archetype 39 (6_D_2_1_ HR_LG_DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A	2	S	1042.0	0.45	621.0	5.21	3.10	1.68
1	2	M	2.5	A	3	S	1563.0	0.73	1071.6	7.82	5.36	1.46

Table B. 90. Archetype 98 (6_E_2_2_ HR_HG_ DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A	2	S	1042.0	0.53	911.5	10.42	9.12	1.14
1	2	M	2.5	A	2	D	2084.0	0.99	1739.1	20.84	17.39	1.20

Table B. 91. Archetype 110 (6_E_2_2_ HR_LG_ DX_SP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
2	2	M	2.5	A	2	S	1042.0	0.52	876.7	10.42	8.78	1.19
1	2	M	2.5	A	3	S	1563.0	0.84	1515.4	15.63	15.14	1.03

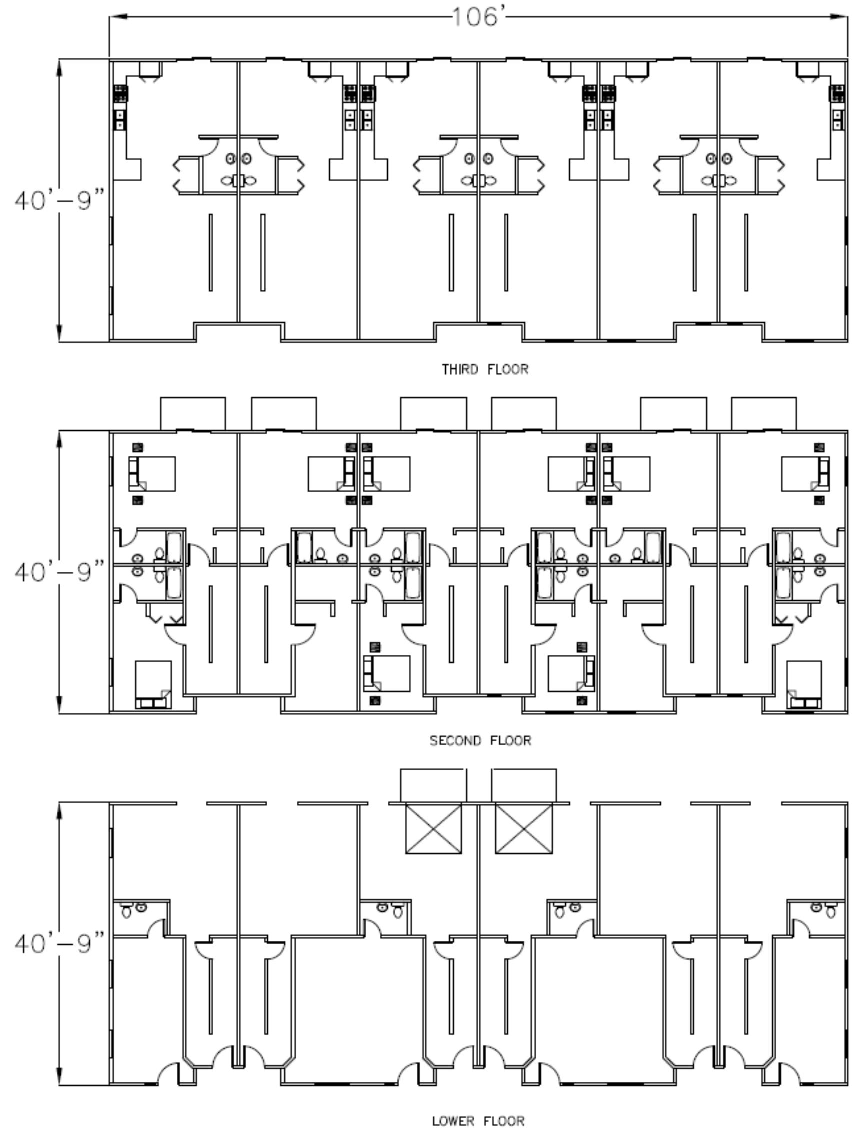


Figure B. 47. Index Bldg. 7 floor plan

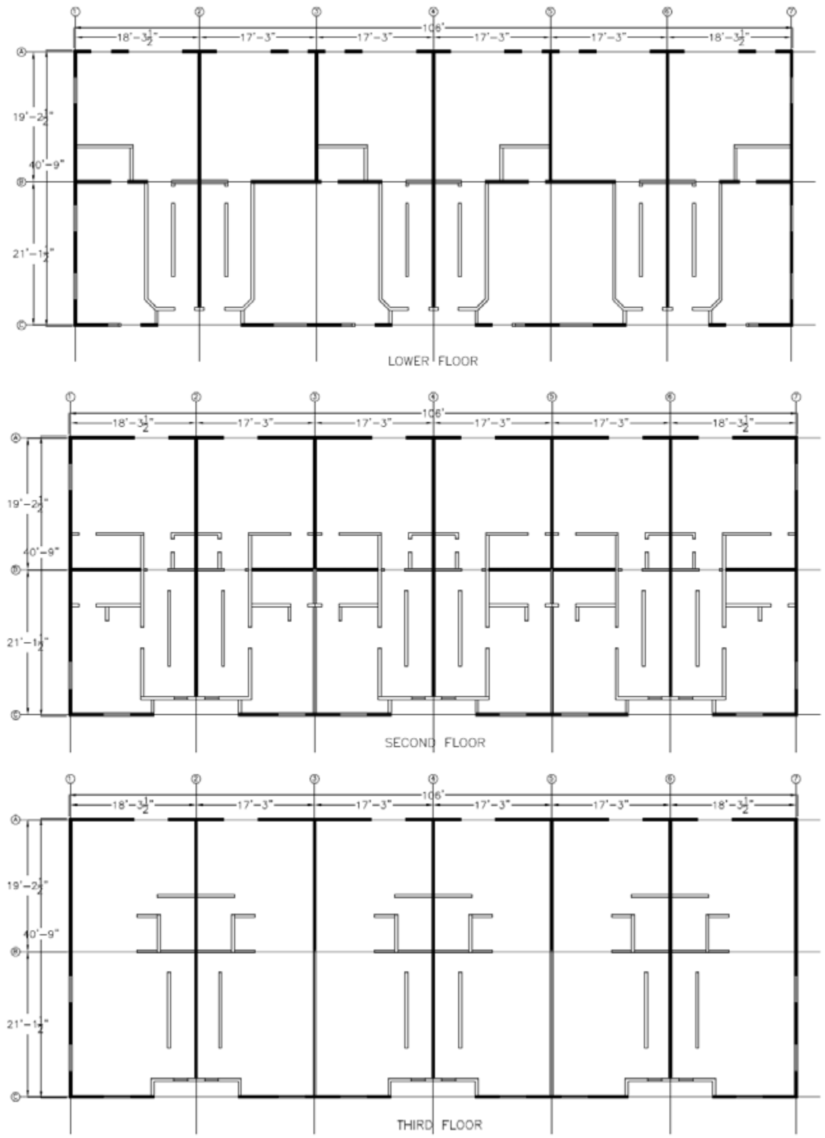


Figure B. 48. Index Bldg. 7 assigned shear walls

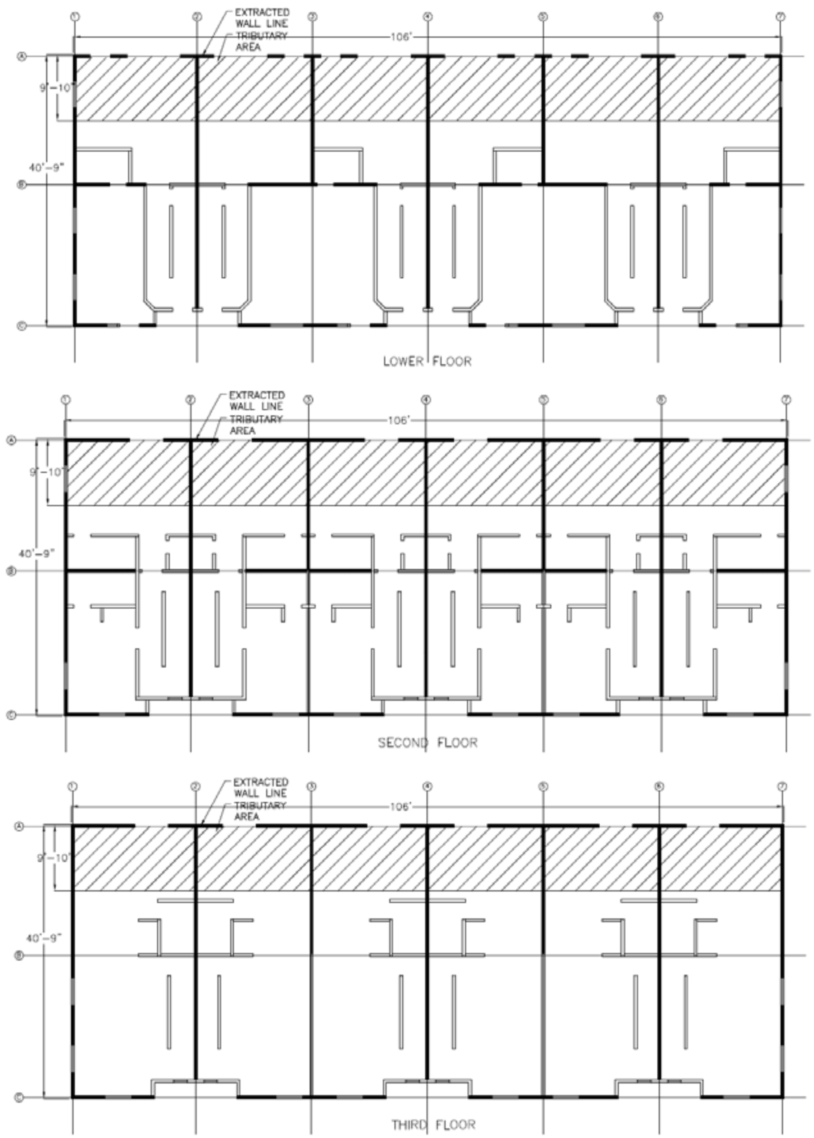


Figure B. 49. Index Bldg. 7 tributary area

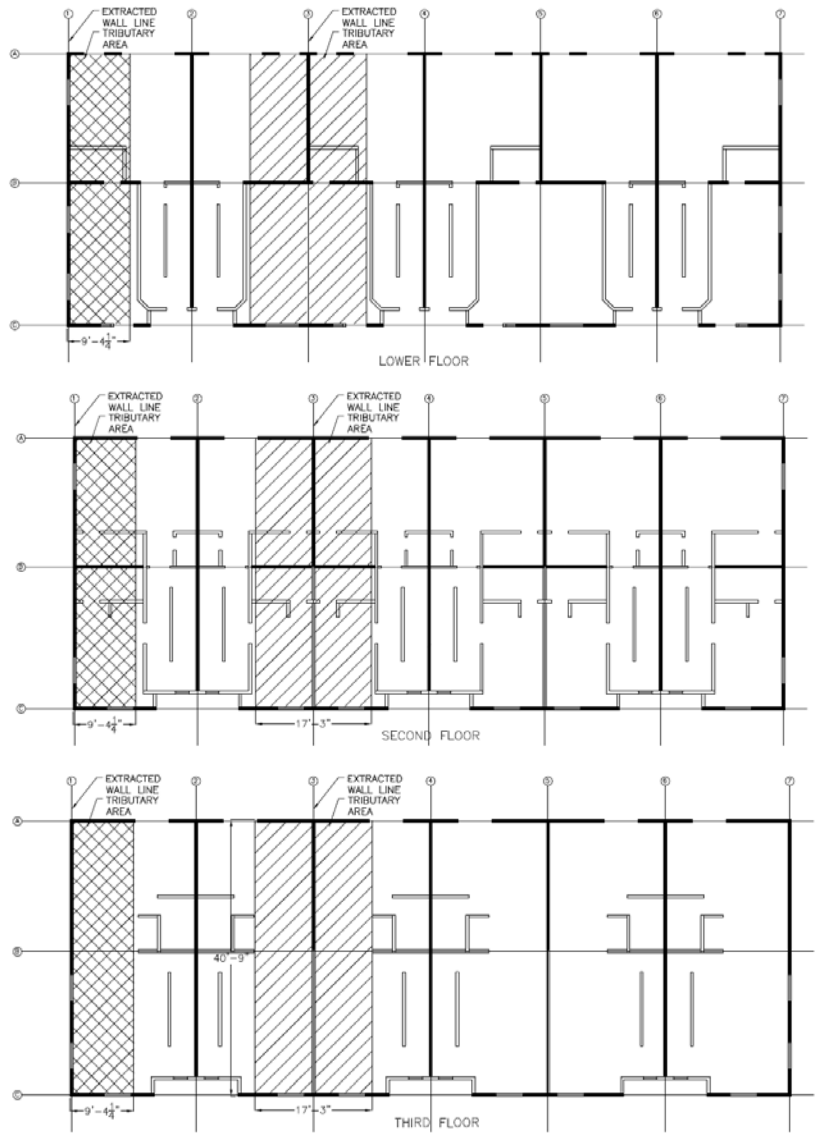


Figure B. 50. Index Bldg. 7 tributary area

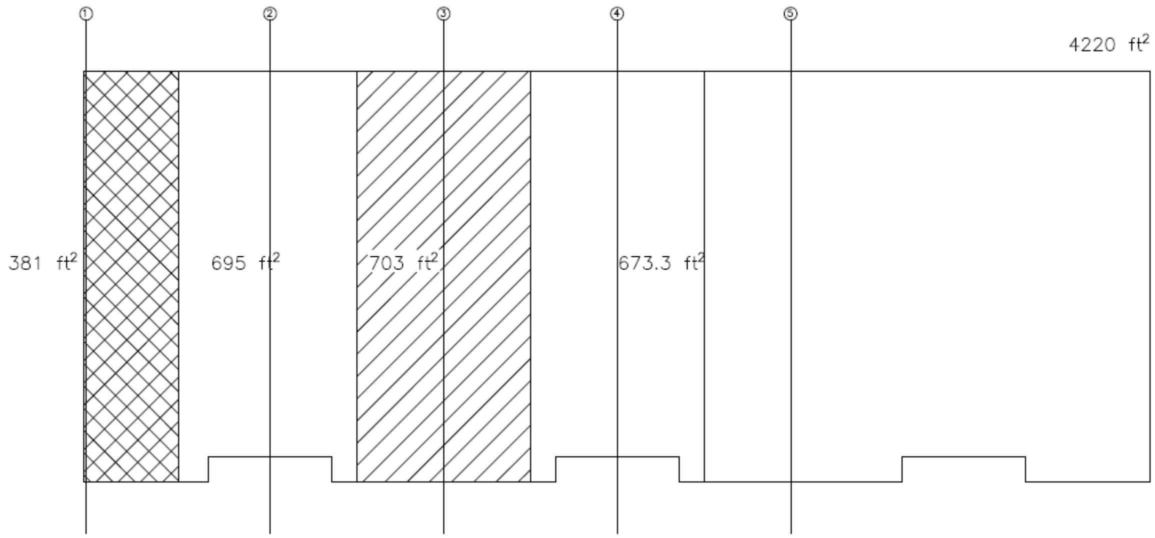


Figure B. 51. Index Bldg. 7 tributary area

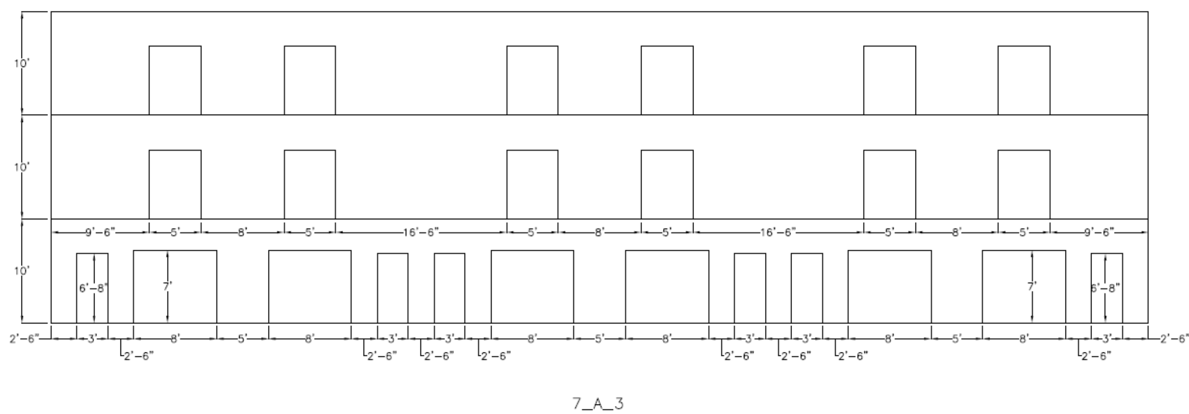


Figure B. 52. Index Bldg. 7 extracted shear wall lines

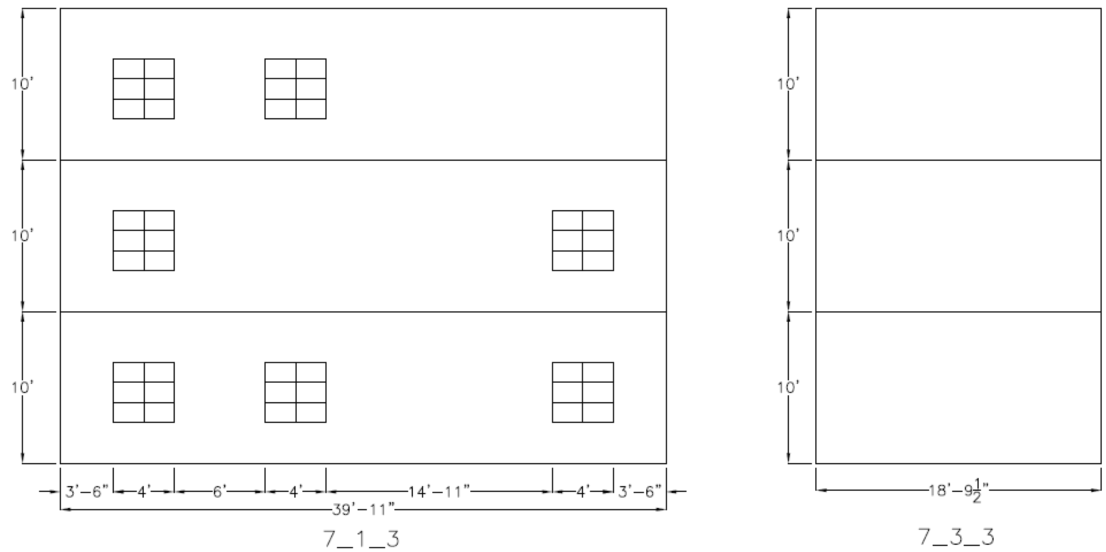


Figure B. 53. Index Bldg. 7 extracted shear wall lines

Table B. 92. Extracted Archetypes, Index Bldg. 7

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

Table B. 93. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 94. Seismic weight summary, Index Bldg. 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				DESIGN SUMMARY
Total Height	h_n=	30.0	ft	Total base shear
Total Weight	W=	822	k	V = 273.97
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		

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 Bldg. 7, 3 story, R=3

INPUT DATA				DESIGN SUMMARY
Total Height	h_n=	30.0	ft	Total base shear
Total Weight	W=	975	k	V = 324.90
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		

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 Bldg. 7, 3 story, R=3

Table B. 95. Tributary load calculation, Index Bldg. 7, 3 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity		High gravity	
				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 B. 96. Tributary load calculation, Index Bldg. 7, 3 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity		High gravity	
				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 B. 97. Archetype 51 (7_3_3_1_MR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	A3	3	D	1563.0	1.18	1396.6	-	-	-
2	1	S	5	A3	8	D	4168.0	2.70	3525.2	-	-	-
1	1	S	5	A3	9	D	4689.0	3.31	4655.3	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
3	4	M	2.5	A3	3	S	1563.0	2.03	1198.1	23.45	18.96	1.24
2	4	M	2.5	A3	3	D	3126.0	3.79	2471.2	52.10	42.34	1.23
1	4	M	2.5	A3	3	D	3126.0	4.36	3070.3	54.71	53.98	1.01

Table B. 98. Archetype 63 (7_3_3_1_MR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	1	S	5	A3	3	D	1563.0	1.03	1233.0	-	-	-
2	1	S	5	A3	6	D	3126.0	2.13	2657.9	-	-	-
1	1	S	5	A3	8	D	4168.0	2.18	3318.0	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
3	4	M	2.5	A3	3	S	1563.0	2.04	1219.7	23.45	18.36	1.28
2	4	M	2.5	A3	3	D	3126.0	3.72	2320.6	46.89	36.50	1.28
1	4	M	2.5	A3	3	D	3126.0	3.80	2892.7	52.10	45.52	1.14

Table B. 99. Archetype 99 (7_A_3_2_ HR_HG_ DX_SP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	1.01	927.9	7.82	6.96	1.12
2	3	M	2.5	A3	2	D	2084.0	1.98	2071.5	15.63	15.54	1.01
1	3	M	2.5	A3	3	D	3126.0	2.77	2657.7	23.45	19.81	1.18

* There are four of (3)2.5ft walls along the wall line and all take equal load.

Table B. 100. Archetype 111 (7_A_3_2_ HR_LG_ DX_SP) design*, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A3	2	S	1042.0	1.00	898.5	7.82	6.74	1.16
2	3	M	2.5	A3	2	D	2084.0	1.88	1785.7	15.63	13.39	1.17
1	3	M	2.5	A3	3	D	3126.0	2.61	2238.9	23.45	16.70	1.40

* There are four of (3)2.5ft walls along the wall line and all take equal load.

INPUT DATA | **DESIGN SUMMARY**

Total Height	h_n=	**30.0**	ft	Total base shear
Total Weight	W=	**822**	k	V = 205.48
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		

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 Bldg. 7, 3 story, R=4

INPUT DATA | **DESIGN SUMMARY**

Total Height	h_n=	**30.0**	ft	Total base shear
Total Weight	W=	**975**	k	V = 243.68
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		

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 Bldg. 7, 3 story, R=4

Table B. 101. Tributary load calculation, Index Bldg. 7, 3 story, E-W direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity		High gravity	
				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 B. 102. Archetype 99 (7_A_3_2_ HR_HG_ DX_SP) design*, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A	2	S	1042.0	0.91	695.9	7.82	5.22	1.50
2	3	M	2.5	A	3	S	1563.0	1.61	1554.9	11.72	11.65	1.01
1	3	M	2.5	A	2	D	2084.0	2.10	1979.9	15.63	14.86	1.05

* There are four of (3)2.5ft walls along the wall line and all take equal load.

Table B. 103. Archetype 111 (7_A_3_2_ HR_LG_ DX_SP) design*, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
3	3	M	2.5	A	2	S	1042.0	0.90	673.8	7.82	5.05	1.55
2	3	M	2.5	A	3	S	1563.0	1.53	1339.2	11.72	10.04	1.17
1	3	M	2.5	A	2	D	2084.0	1.98	1671.9	15.63	12.53	1.25

* There are four of (3)2.5ft walls along the wall line and all take equal load.

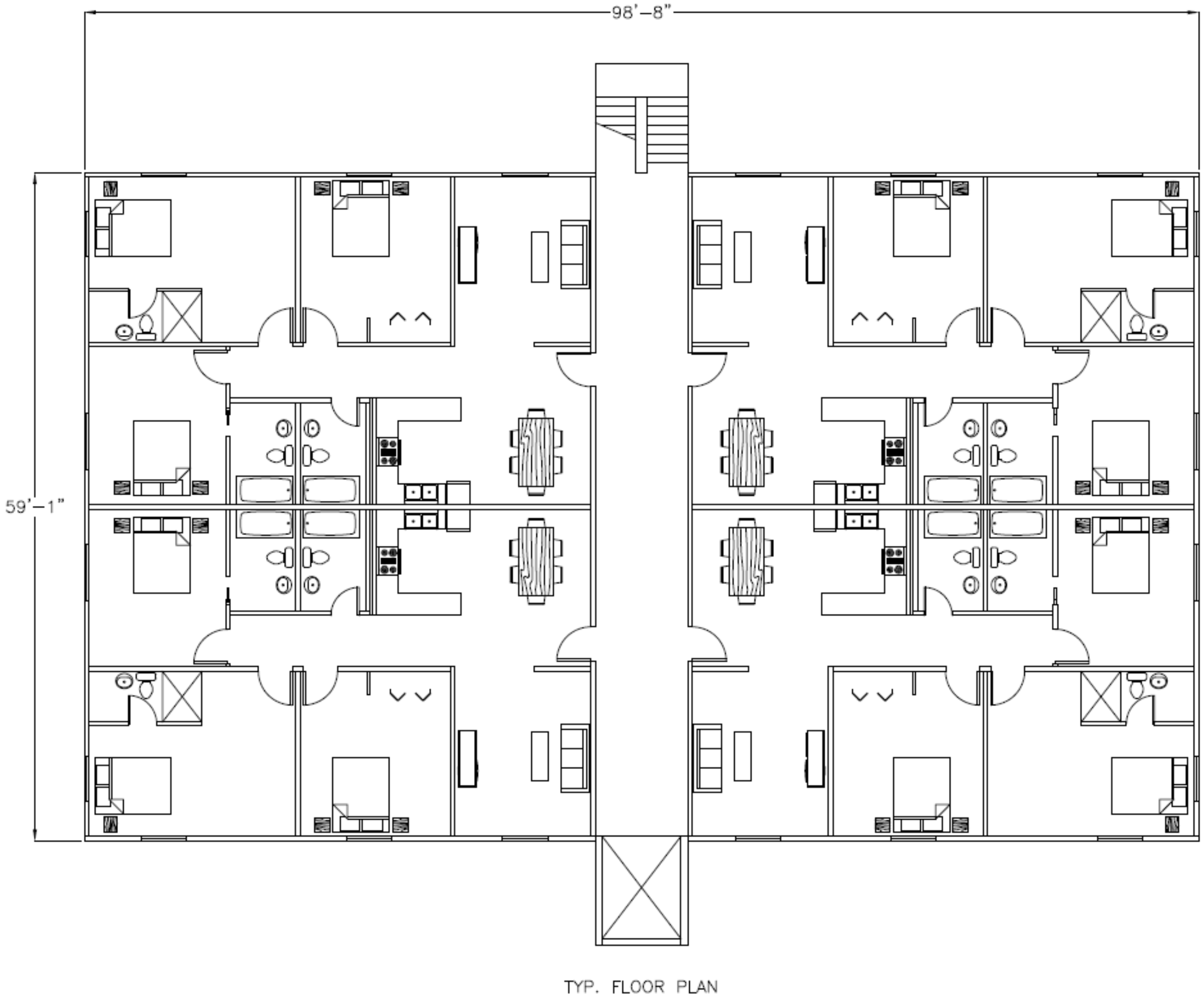


Figure B. 54. Index Bldg. 8 floor plan

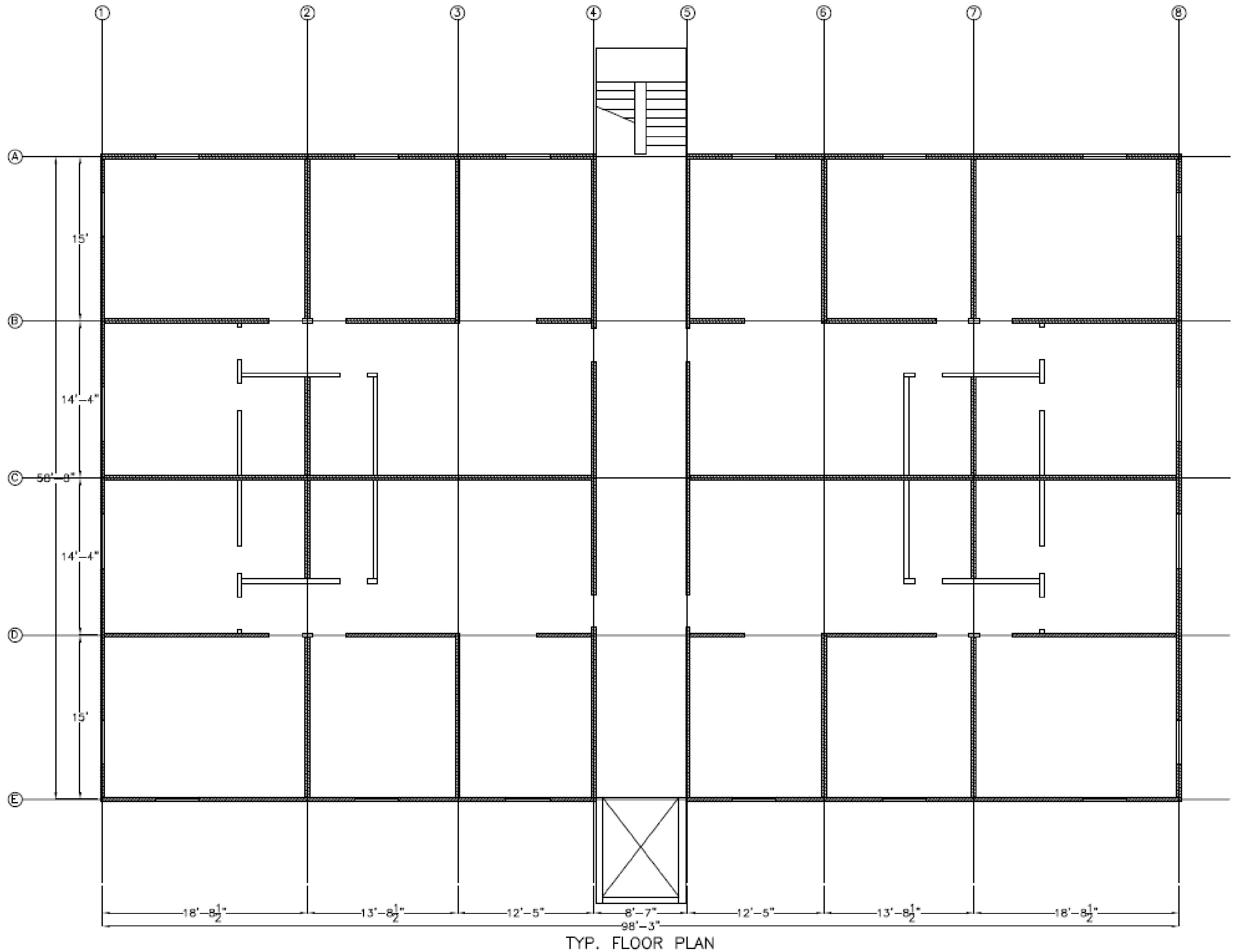


Figure B. 55. Index Bldg. 8 assigned shear walls

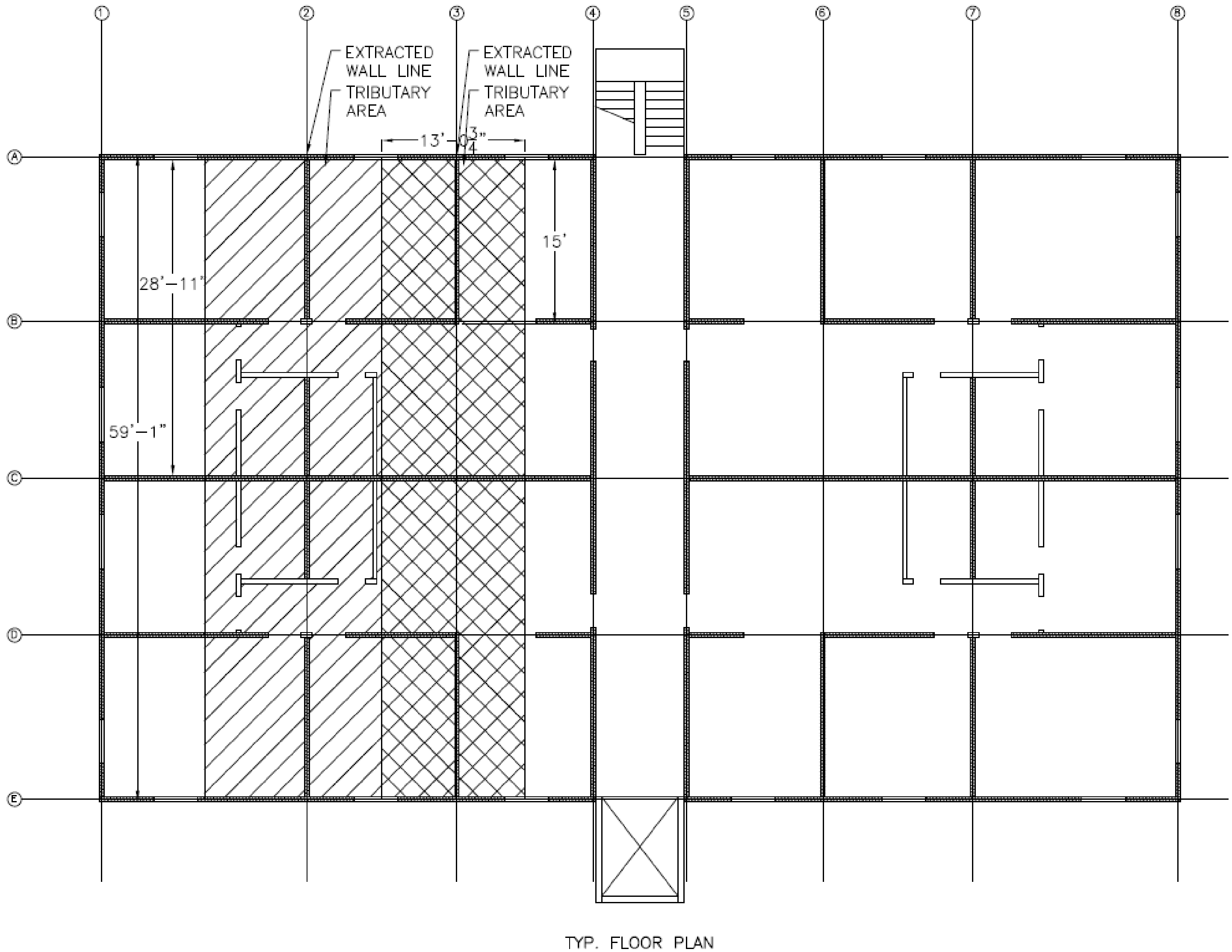


Figure B. 56. Index Bldg. 8 tributary area

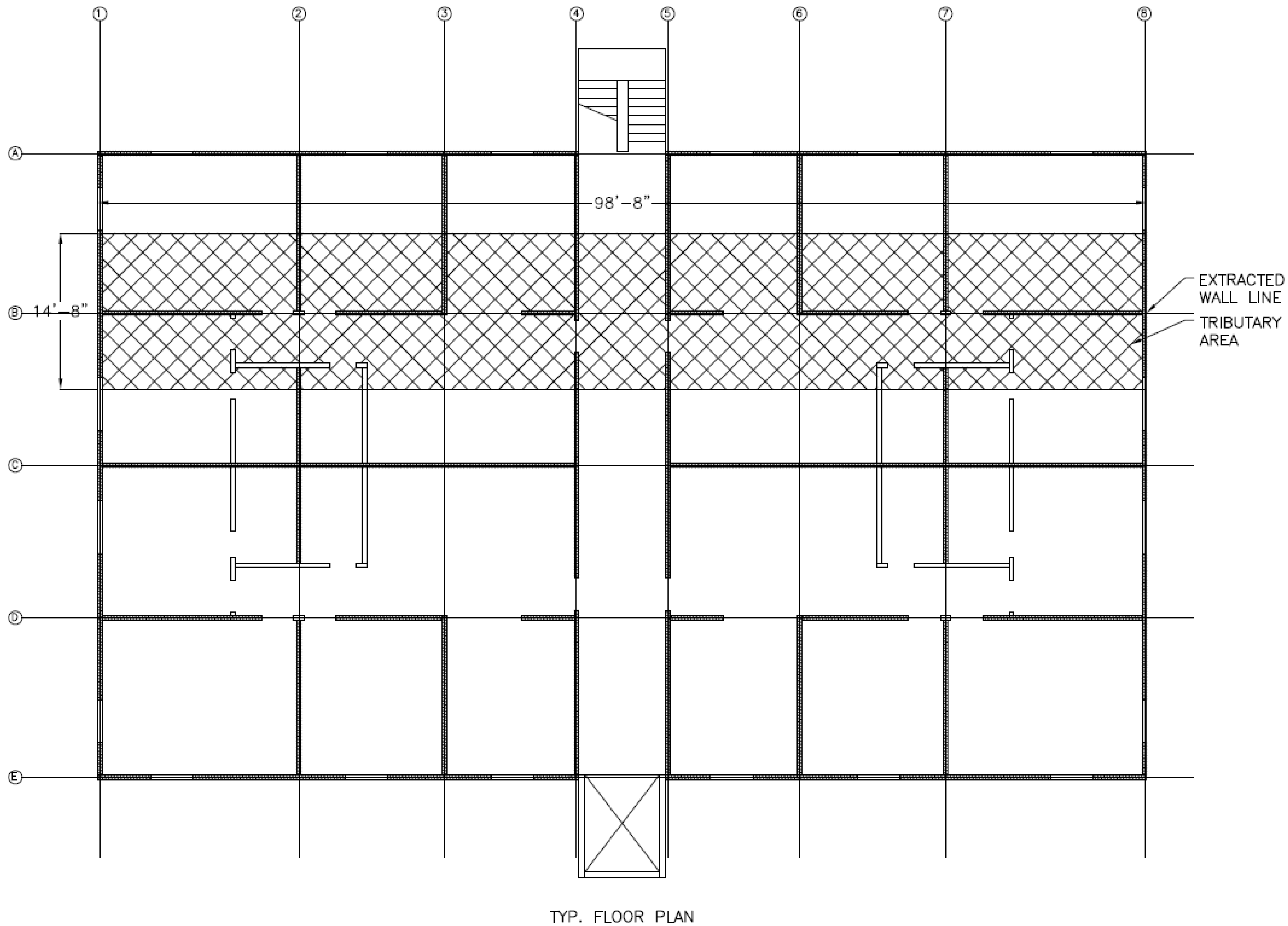


Figure B. 57. Index Bldg. 8 tributary area

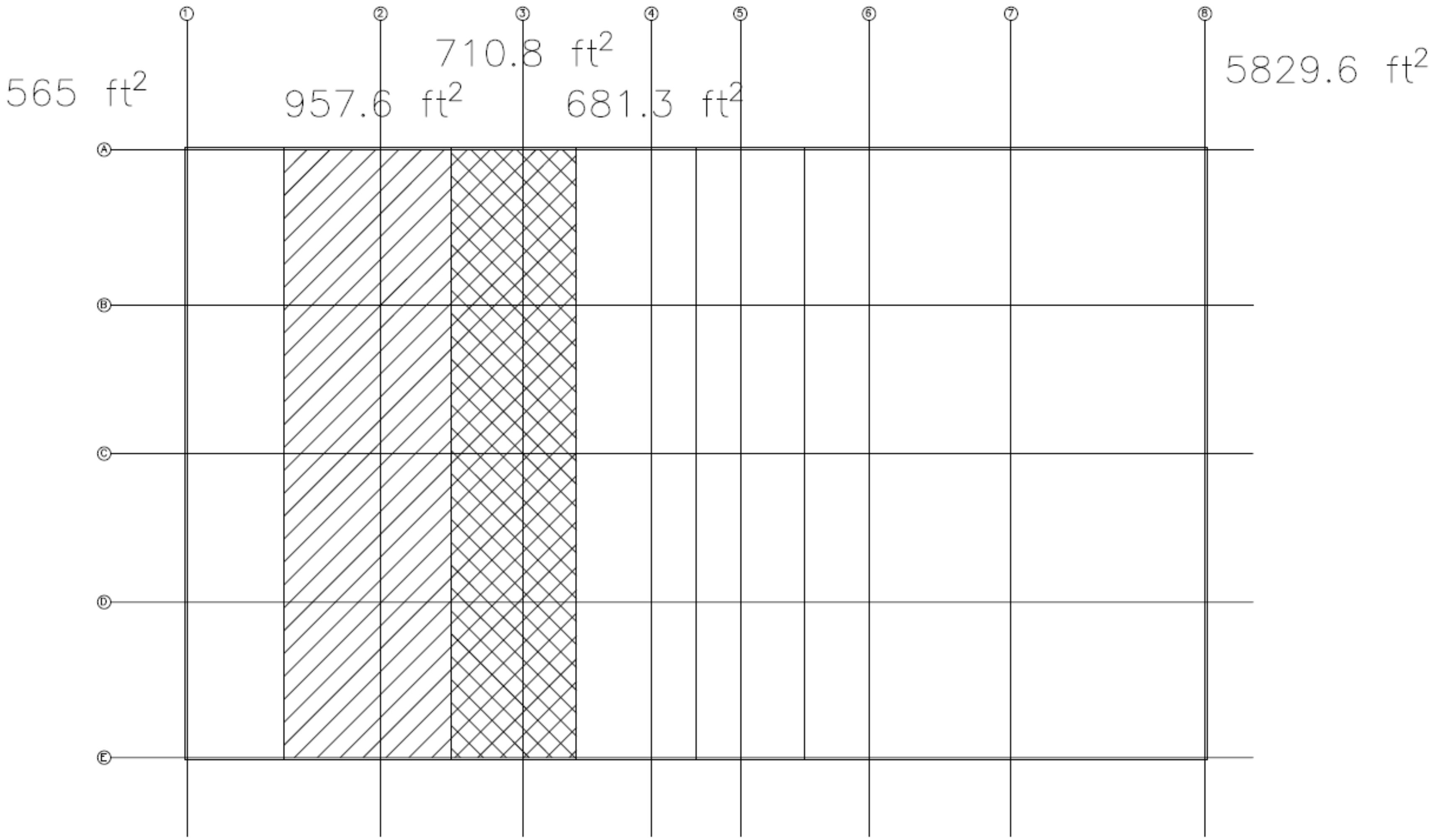


Figure B. 58. Index Bldg. 8 tributary area

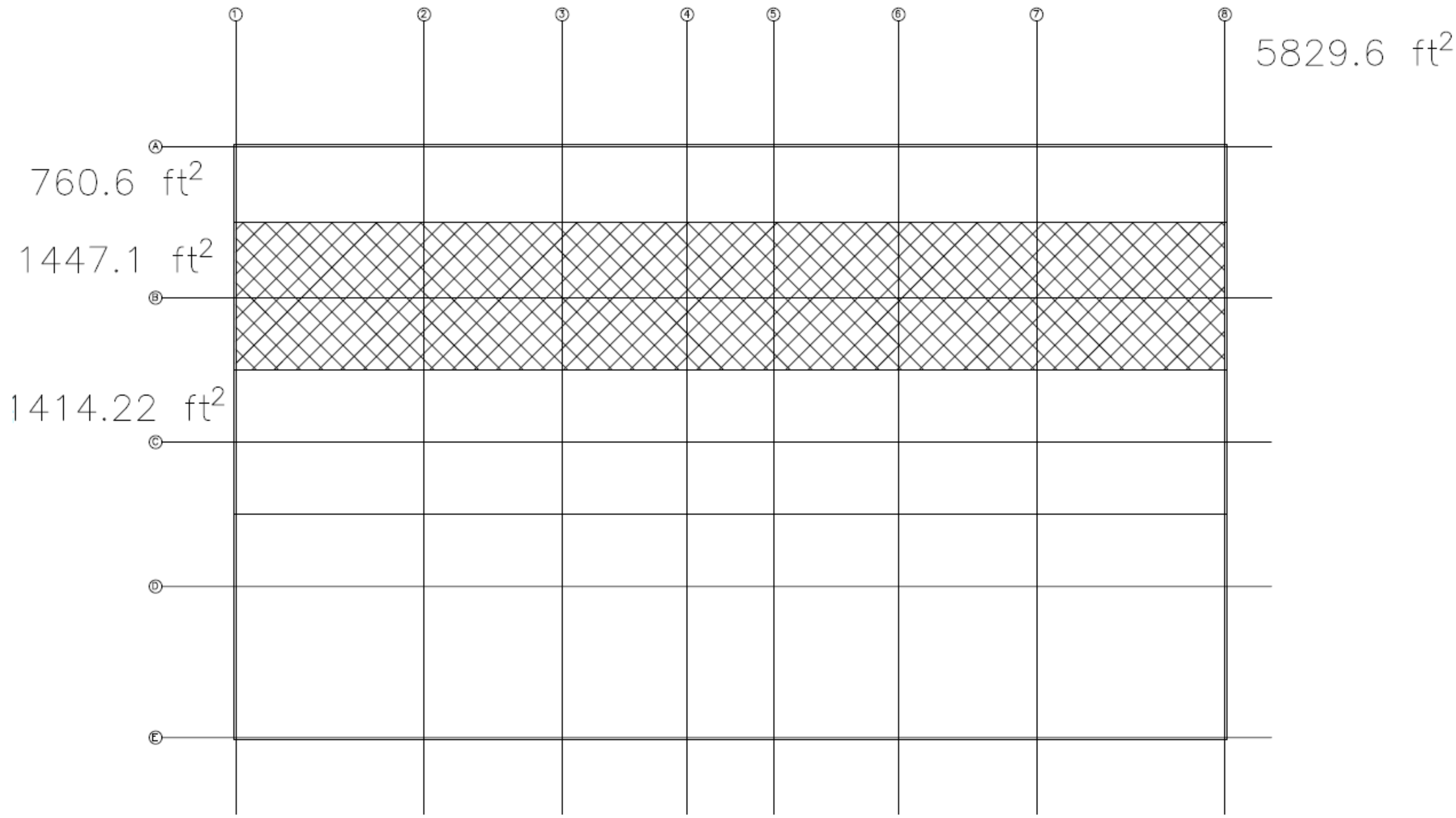


Figure B. 59. Index Bldg. 8 tributary area

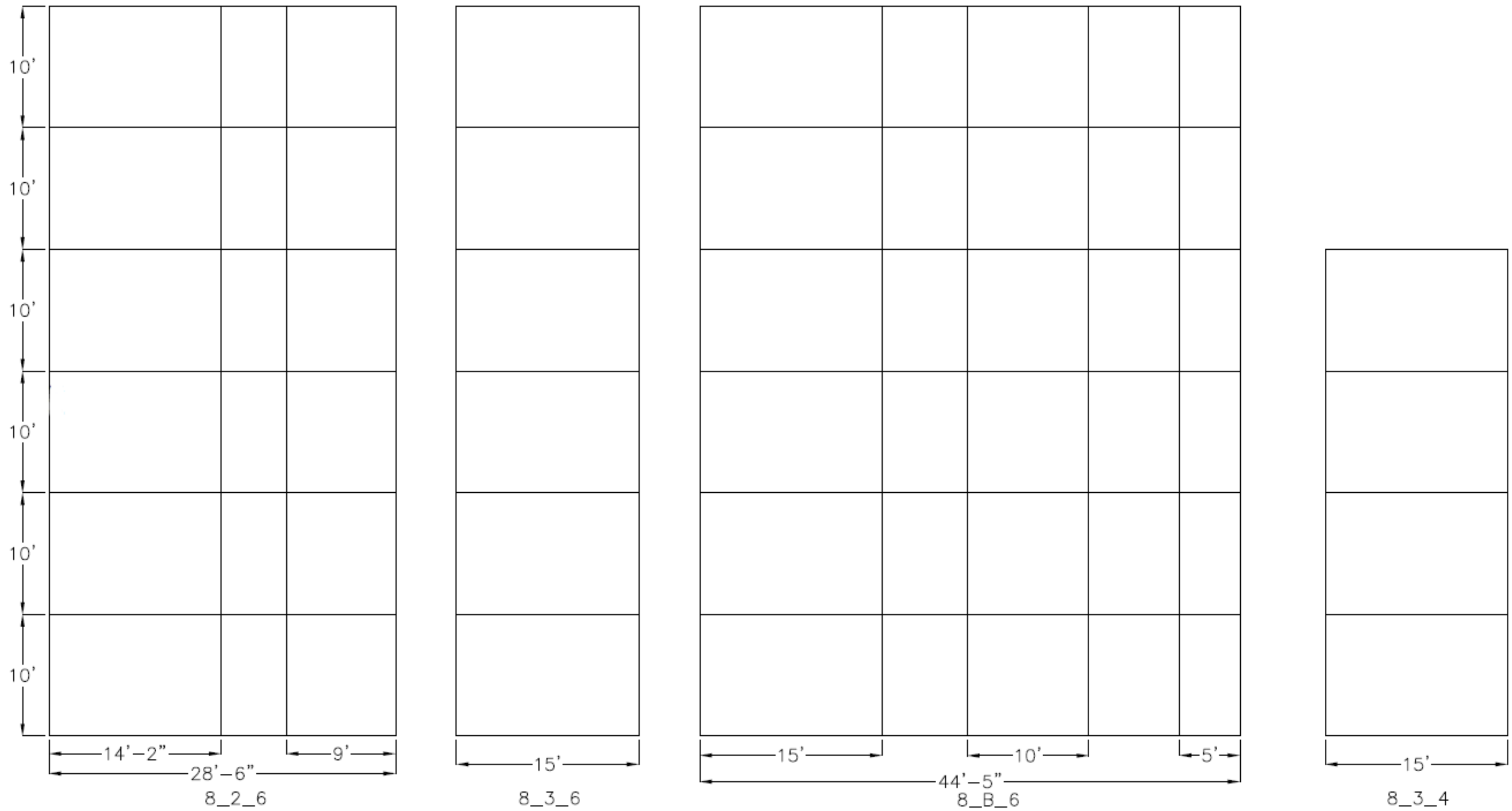


Figure B. 60. Index Bldg. 8 extracted shear wall lines

Table B. 104. Extracted Archetypes, Index Bldg. 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 B. 105. Seismic weight detailed weight calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 106. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 107. Seismic weight summary, Index Bldg. 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 B. 108. Seismic weight summary, Index Bldg. 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							DESIGN SUMMARY
Total Height	h_n=	**40.0**	ft				Total base shear
Total Weight	W=	**1,583**	k				V = 395.80
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					

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 Bldg. 8, 4 story, R=3

INPUT DATA							DESIGN SUMMARY
Total Height	h_n=	**40.0**	ft				Total base shear
Total Weight	W=	**1,900**	k				V = 475.00
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					

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 Bldg. 8, 4 story, R=3

INPUT DATA				DESIGN SUMMARY		
Total Height	h_n=	60.0	ft	Total base shear		
Total Weight	W=	2,442	k	V	=	809.24
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				

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 Bldg. 8, 6 story, R=3

INPUT DATA				DESIGN SUMMARY		
Total Height	h_n=	60.0	ft	Total base shear		
Total Weight	W=	2,970	k	V	=	984.18
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				

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 Bldg. 8, 6 story, R=3

Table B. 109. Tributary load calculation, Index Bldg. 8, 4 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)	High gravity Story shear (kip)	High gravity 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 B. 110. Tributary load calculation, Index Bldg. 8, 4 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)	High gravity Story shear (kip)	High gravity 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 B. 111. Tributary load calculation, Index Bldg. 8, 6 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 112. Tributary load calculation, Index Bldg. 8, 6 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 113. Archetype 5 (8_3_6_1_ LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	2	S	521.0	2.17	369.5	7.82	5.54	1.41
5	3	M	5	A3	4	S	1042.0	4.43	924.0	15.63	13.86	1.13
4	3	M	5	A3	3	D	1563.0	6.34	1362.5	23.45	20.44	1.15
3	3	M	5	A3	4	D	2084.0	8.26	1686.5	31.26	25.30	1.24
2	3	M	5	A3	4	D	2084.0	8.94	1898.0	31.26	28.47	1.10
1	3	M	5	A3	4	D	2084.0	9.28	2000.1	31.26	30.00	1.04

Table B. 114. Archetype 17 (8_3_6_1_ LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	2	S	521.0	2.16	362.1	7.82	5.43	1.44
5	3	M	5	A3	4	S	1042.0	4.19	798.2	15.63	11.97	1.31
4	3	M	5	A3	5	S	1302.5	5.50	1143.1	19.54	17.15	1.14
3	3	M	5	A3	6	S	1563.0	6.35	1397.9	23.45	20.97	1.12
2	3	M	5	A3	7	S	1823.5	7.49	1564.3	27.35	23.46	1.17
1	3	M	5	A3	7	S	1823.5	7.59	1644.6	27.35	24.67	1.11

Table B. 115. Archetype 29 (8_3_6_1_ HR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.07	369.5	15.63	5.54	2.82
5	6	M	2.5	A3	2	S	1042.0	3.08	924.0	15.63	13.86	1.13
4	6	M	2.5	A3	2	D	2084.0	5.37	1362.5	31.26	20.44	1.53
3	6	M	2.5	A3	2	D	2084.0	6.08	1686.5	31.26	25.30	1.24
2	6	M	2.5	A3	2	D	2084.0	6.25	1898.0	31.26	28.47	1.10
1	6	M	2.5	A3	2	D	2084.0	6.64	2000.1	31.26	30.00	1.04

Table B. 116. Archetype 41 (8_3_6_1_ HR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.05	362.1	15.63	5.43	2.88
5	6	M	2.5	A3	2	S	1042.0	2.96	798.2	15.63	11.97	1.31
4	6	M	2.5	A3	3	S	1563.0	4.27	1143.1	23.45	17.15	1.37
3	6	M	2.5	A3	3	S	1563.0	4.72	1397.9	23.45	20.97	1.12
2	6	M	2.5	A3	2	D	2084.0	5.93	1564.3	31.26	23.46	1.33
1	6	M	2.5	A3	2	D	2084.0	5.97	1644.6	31.26	24.67	1.27

Table B. 117. Archetype 53 (8_3_6_1_ MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.13	100.8	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.24	198.7	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.25	213.5	-	-	-
3	2	M	2.5	A3	2	S	1042.0	0.27	226.3	-	-	-
2	2	M	2.5	A3	2	S	1042.0	0.26	208.6	-	-	-
1	2	M	2.5	A3	2	S	1042.0	0.27	218.1	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	A3	2	S	521.0	1.31	503.8	10.42	5.54	1.88
5	2	M	5	A3	5	S	1302.5	3.08	1286.7	18.24	13.86	1.32
4	2	M	5	A3	4	D	2084.0	4.63	1937.0	26.05	20.44	1.27
3	2	M	5	A3	5	D	2605.0	5.74	2416.6	31.26	25.30	1.24
2	2	M	5	A3	6	D	3126.0	6.71	2742.8	36.47	28.47	1.28
1	2	M	5	A3	6	D	3126.0	7.06	2891.0	36.47	30.00	1.22

Table B. 118. Archetype 65 (8_3_6_1_ MR_LG_ DX_LP design), R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.24	467.9	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.72	843.9	-	-	-
4	4	M	2.5	A3	3	S	1563.0	2.56	1344.1	-	-	-
3	4	M	2.5	A3	2	D	2084.0	3.34	1740.7	-	-	-
2	4	M	2.5	A3	2	D	2084.0	3.59	1968.2	-	-	-
1	4	M	2.5	A3	2	D	2084.0	3.71	2034.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	2	S	521.0	0.20	150.4	13.03	5.43	2.40
5	1	S	5	A3	3	S	781.5	0.72	706.7	14.33	11.97	1.20
4	1	S	5	A3	3	S	781.5	0.71	741.1	19.54	17.15	1.14
3	1	S	5	A3	3	S	781.5	0.68	712.4	24.75	20.97	1.18
2	1	S	5	A3	3	S	781.5	0.69	756.6	24.75	23.46	1.05
1	1	S	5	A3	4	S	1042.0	0.79	864.0	26.05	24.67	1.06

Table B. 119. Archetype 49 (8_3_4_1_MR_HG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	A3	2	S	1042.0	0.96	724.2	-	-	-
3	3	M	2.5	A3	2	D	2084.0	1.88	1593.6	-	-	-
2	3	M	2.5	A3	3	D	3126.0	2.60	2266.3	-	-	-
1	3	M	2.5	A3	3	D	3126.0	3.14	2996.9	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
4	1	S	5	A3	4	S	1042.0	0.87	985.7	13.03	10.36	1.26
3	1	S	5	A3	5	D	2605.0	1.97	2506.8	28.66	24.49	1.17
2	1	S	5	A3	7	D	3647.0	2.59	3381.3	41.68	33.90	1.23
1	1	S	5	A3	7	D	3647.0	2.25	3227.1	41.68	38.61	1.08

Table B. 120. Archetype 61 (8_3_4_1_MR_LG_DX_SP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
4	3	M	2.5	A3	2	S	1042.0	0.92	692.6	-	-	-
3	3	M	2.5	A3	2	S	1042.0	1.17	1012.6	-	-	-
2	3	M	2.5	A3	2	D	2084.0	2.02	1745.8	-	-	-
1	3	M	2.5	A3	2	D	2084.0	2.32	2032.5	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
4	1	S	5	A3	4	S	1042.0	0.87	979.0	13.03	10.09	1.29
3	1	S	5	A3	6	D	3126.0	2.09	2707.5	23.45	21.13	1.11
2	1	S	5	A3	6	D	3126.0	2.38	3080.0	31.26	28.49	1.10
1	1	S	5	A3	7	D	3647.0	2.58	3386.0	33.87	32.17	1.05

Table B. 121. Archetype 77 (8_B_6_2_ LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	A3	2	S	521.0	1.07	345.9	-	-	-
5	2	M	5	A3	3	S	781.5	1.53	519.7	-	-	-
4	2	M	5	A3	4	S	1042.0	1.96	694.6	-	-	-
3	2	M	5	A3	3	D	1563.0	3.31	1288.3	-	-	-
2	2	M	5	A3	3	D	1563.0	3.23	1224.1	-	-	-
1	2	M	5	A3	3	D	1563.0	2.93	1006.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.05	30.5	-	-	-
5	1	S	5	A3	3	S	781.5	0.06	42.7	-	-	-
4	1	S	5	A3	3	S	781.5	0.13	90.2	-	-	-
3	1	S	5	A3	3	S	781.5	0.27	213.8	-	-	-
2	1	S	5	A3	3	S	781.5	0.20	152.5	-	-	-
1	1	S	5	A3	3	S	781.5	0.06	38.0	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	2	S	521.0	2.38	515.9	16.93	11.35	1.49
5	3	M	5	A3	3	D	1563.0	6.78	1537.6	35.17	28.47	1.24
4	3	M	5	A3	5	D	2605.0	9.77	2308.0	53.40	42.02	1.27
3	3	M	5	A3	5	D	2605.0	9.77	2538.0	58.61	52.02	1.13
2	3	M	5	A3	6	D	3126.0	12.01	3036.7	66.43	58.55	1.13
1	3	M	5	A3	7	D	3647.0	14.97	3430.2	74.24	61.71	1.20

Table B. 122. Archetype 89 (8_B_6_2_ LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	2	S	521.0	0.00	1.2	-	-	-
5	1	S	5	A3	2	S	521.0	0.05	34.4	-	-	-
4	1	S	5	A3	2	S	521.0	0.10	68.8	-	-	-
3	1	S	5	A3	2	S	521.0	0.15	110.9	-	-	-
2	1	S	5	A3	2	S	521.0	0.10	68.4	-	-	-
1	1	S	5	A3	2	S	521.0	0.07	46.0	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
6	2	M	5	A3	2	S	521.0	1.06	340.6	-	-	-
5	2	M	5	A3	3	S	781.5	1.54	525.6	-	-	-
4	2	M	5	A3	3	S	781.5	1.60	568.3	-	-	-
3	2	M	5	A3	4	S	1042.0	2.07	747.8	-	-	-
2	2	M	5	A3	4	S	1042.0	2.04	709.2	-	-	-
1	2	M	5	A3	4	S	1042.0	2.07	697.9	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
6	3	M	5	A3	2	S	521.0	2.38	509.6	15.63	11.06	1.41
5	3	M	5	A3	5	S	1302.5	5.55	1263.1	29.96	24.37	1.23
4	3	M	5	A3	4	D	2084.0	8.14	1925.3	41.68	34.91	1.19
3	3	M	5	A3	5	D	2605.0	9.59	2310.4	52.10	42.69	1.22
2	3	M	5	A3	6	D	3126.0	11.62	2689.0	59.92	47.77	1.25
1	3	M	5	A3	6	D	3126.0	12.73	2867.4	59.92	50.22	1.19

Table B. 123. Archetype 101 (8_B_6_2_ HR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.02	14.2	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.39	361.0	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.37	359.2	-	-	-
3	2	M	2.5	A3	2	S	1042.0	0.40	376.2	-	-	-
2	2	M	2.5	A3	2	S	1042.0	0.35	341.6	-	-	-
1	2	M	2.5	A3	2	S	1042.0	0.35	335.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.31	510.7	-	-	-
5	6	M	2.5	A3	3	S	1563.0	4.03	1258.9	-	-	-
4	6	M	2.5	A3	2	D	2084.0	5.44	1756.0	-	-	-
3	6	M	2.5	A3	3	D	3126.0	7.50	2358.5	-	-	-
2	6	M	2.5	A3	3	D	3126.0	8.39	2706.0	-	-	-
1	6	M	2.5	A3	3	D	3126.0	9.03	2896.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.09	361.8	31.26	11.35	2.75
5	4	M	2.5	A3	2	S	1042.0	1.66	778.6	39.08	28.47	1.37
4	4	M	2.5	A3	2	D	2084.0	2.87	1388.0	57.31	42.02	1.36
3	4	M	2.5	A3	2	D	2084.0	3.13	1476.5	72.94	52.02	1.40
2	4	M	2.5	A3	2	D	2084.0	3.36	1625.7	72.94	58.55	1.25
1	4	M	2.5	A3	2	D	2084.0	3.44	1657.7	72.94	61.71	1.18

Table B. 124. Archetype 113 (8_B_6_2_ HR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.01	5.6	-	-	-
5	2	M	2.5	A3	2	S	1042.0	0.37	367.4	-	-	-
4	2	M	2.5	A3	2	S	1042.0	0.48	508.6	-	-	-
3	2	M	2.5	A3	2	S	1042.0	0.50	526.4	-	-	-
2	2	M	2.5	A3	2	S	1042.0	0.44	447.0	-	-	-
1	2	M	2.5	A3	2	S	1042.0	0.47	471.6	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.31	505.7	-	-	-
5	6	M	2.5	A3	2	S	1042.0	2.90	949.9	-	-	-
4	6	M	2.5	A3	3	S	1563.0	4.16	1479.6	-	-	-
3	6	M	2.5	A3	2	D	2084.0	5.64	1976.8	-	-	-
2	6	M	2.5	A3	2	D	2084.0	5.52	1856.3	-	-	-
1	6	M	2.5	A3	2	D	2084.0	5.87	1966.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.05	344.2	31.26	11.06	2.83
5	4	M	2.5	A3	2	S	1042.0	1.69	828.9	31.26	24.37	1.28
4	4	M	2.5	A3	2	S	1042.0	1.91	1016.9	39.08	34.91	1.12
3	4	M	2.5	A3	2	S	1042.0	1.98	1040.6	46.89	42.69	1.10
2	4	M	2.5	A3	2	D	2084.0	3.51	1769.0	57.31	47.77	1.20
1	4	M	2.5	A3	2	D	2084.0	3.65	1836.8	57.31	50.22	1.14

Table B. 125. Archetype 125 (8_B_6_2_ MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.08	52.1	-	-	-
5	1	S	5	A3	3	S	781.5	0.48	442.0	-	-	-
4	1	S	5	A3	3	S	781.5	0.29	249.0	-	-	-
3	1	S	5	A3	3	S	781.5	0.50	461.0	-	-	-
2	1	S	5	A3	3	S	781.5	0.41	371.3	-	-	-
1	1	S	5	A3	4	S	1042.0	0.60	560.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	A3	2	S	521.0	1.08	353.5	-	-	-
5	2	M	5	A3	6	S	1563.0	3.22	1472.7	-	-	-
4	2	M	5	A3	5	D	2605.0	5.12	2161.0	-	-	-
3	2	M	5	A3	6	D	3126.0	6.06	2820.1	-	-	-
2	2	M	5	A3	7	D	3647.0	7.19	3251.2	-	-	-
1	2	M	5	A3	7	D	3647.0	7.17	3379.2	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.30	503.6	24.75	11.35	2.18
5	6	M	2.5	A3	2	S	1042.0	2.52	769.1	35.17	28.47	1.24
4	6	M	2.5	A3	2	D	2084.0	4.54	1277.4	61.22	42.02	1.46
3	6	M	2.5	A3	2	D	2084.0	4.63	1434.4	66.43	52.02	1.28
2	6	M	2.5	A3	2	D	2084.0	5.35	1612.4	71.64	58.55	1.22
1	6	M	2.5	A3	2	D	2084.0	5.33	1674.0	72.94	61.71	1.18

Table B. 126. Archetype 137 (8_B_6_2_ MR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	1	S	5	A3	3	S	781.5	0.03	21.6	-	-	-
5	1	S	5	A3	3	S	781.5	0.58	540.3	-	-	-
4	1	S	5	A3	3	S	781.5	0.54	503.8	-	-	-
3	1	S	5	A3	3	S	781.5	0.48	441.8	-	-	-
2	1	S	5	A3	3	S	781.5	0.55	522.1	-	-	-
1	1	S	5	A3	3	S	781.5	0.59	572.1	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
6	4	M	2.5	A3	2	S	1042.0	1.04	341.1	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.63	756.0	-	-	-
4	4	M	2.5	A3	2	S	1042.0	1.74	806.9	-	-	-
3	4	M	2.5	A3	2	D	2084.0	2.98	1374.6	-	-	-
2	4	M	2.5	A3	2	D	2084.0	3.34	1576.9	-	-	-
1	4	M	2.5	A3	2	D	2084.0	3.38	1642.2	-	-	-
No. of Stories	**No. of Panels**	**Configuration**	**Panel Length (ft)**	**Connector type**	**NC, Number of Connectors/Side/Panel**	**S/D**	**Shear Capacity (plf)**	**Stiffness (kip/in.)**	**Applied Load (plf)**	**Shear Strength Provided (kip)**	**Story Shear (kip)/Archetype**	**Ratio of Provided Shear to Story Shear**
6	6	M	2.5	A3	2	S	1042.0	2.30	502.4	29.96	11.06	2.71
5	6	M	2.5	A3	2	S	1042.0	3.04	940.9	29.96	24.37	1.23
4	6	M	2.5	A3	2	D	2084.0	5.26	1621.2	45.59	34.91	1.31
3	6	M	2.5	A3	2	D	2084.0	5.79	1782.2	56.01	42.69	1.31
2	6	M	2.5	A3	2	D	2084.0	6.23	1959.3	56.01	47.77	1.17
1	6	M	2.5	A3	2	D	2084.0	6.37	2062.5	56.01	50.22	1.12

INPUT DATA				DESIGN SUMMARY			
Total Height	h_n=	60.0	ft	Total base shear			
Total Weight	W=	2,442	k		V	=	606.93
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					

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 Bldg. 8, 6 story, R=4

INPUT DATA				DESIGN SUMMARY			
Total Height	h_n=	60.0	ft	Total base shear			
Total Weight	W=	2,970	k		V	=	738.13
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					

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 Bldg. 8, 6 story, R=4

Table B. 127. Tributary load calculation, Index Bldg. 8, 6 story, N-S direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: 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 B. 128. Tributary load calculation, Index Bldg. 8, 6 story, E-W direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity Story shear (kip)	Low gravity Cumulative Shear Load (kip)*	High gravity Story shear (kip)	High gravity 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 B. 129. Archetype 29 (8_3_6_1_ HR_HG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	1.83	277.1	15.63	4.16	3.76
5	6	M	2.5	A3	2	S	1042.0	2.86	693.0	15.63	10.40	1.50
4	6	M	2.5	A3	2	S	1042.0	3.49	1021.9	15.63	15.33	1.02
3	6	M	2.5	A3	3	S	1563.0	4.82	1264.9	23.45	18.97	1.24
2	6	M	2.5	A3	3	S	1563.0	4.96	1423.5	23.45	21.35	1.10
1	6	M	2.5	A3	3	S	1563.0	5.18	1500.1	23.45	22.50	1.04

Table B. 130. Archetype 41 (8_3_6_1_ HR_LG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	1.81	271.5	15.63	4.07	3.84
5	6	M	2.5	A3	2	S	1042.0	2.72	598.6	15.63	8.98	1.74
4	6	M	2.5	A3	2	S	1042.0	3.20	857.3	15.63	12.86	1.22
3	6	M	2.5	A3	3	S	1563.0	4.37	1048.5	23.45	15.73	1.49
2	6	M	2.5	A3	3	S	1563.0	4.69	1173.2	23.45	17.60	1.33
1	6	M	2.5	A3	3	S	1563.0	4.76	1233.4	23.45	18.50	1.27

Table B. 131. Archetype 101 (8_B_6_2_ HR_HG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.10	391.6	-	-	-
5	6	M	2.5	A3	2	S	1042.0	3.00	873.3	-	-	-
4	6	M	2.5	A3	2	D	2084.0	5.18	1514.2	-	-	-
3	6	M	2.5	A3	2	D	2084.0	5.90	1870.2	-	-	-
2	6	M	2.5	A3	3	D	3126.0	7.98	2298.7	-	-	-
1	6	M	2.5	A3	3	D	3126.0	8.46	2434.9	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	0.89	248.1	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.58	691.1	-	-	-
4	4	M	2.5	A3	2	S	1042.0	1.66	728.0	-	-	-
3	4	M	2.5	A3	2	S	1042.0	1.85	880.7	-	-	-
2	4	M	2.5	A3	2	S	1042.0	1.78	768.6	-	-	-
1	4	M	2.5	A3	2	S	1042.0	1.84	795.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.04	21.4	31.26	8.46	3.69
5	2	M	2.5	A3	2	S	1042.0	0.26	230.4	31.26	21.16	1.48
4	2	M	2.5	A3	2	S	1042.0	0.28	242.3	46.89	31.21	1.50
3	2	M	2.5	A3	2	S	1042.0	0.37	353.4	46.89	38.63	1.21
2	2	M	2.5	A3	2	S	1042.0	0.30	260.9	62.52	43.47	1.44
1	2	M	2.5	A3	2	S	1042.0	0.31	265.4	62.52	45.81	1.36

Table B. 132. Archetype 113 (8_B_6_2_ HR_LG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	6	M	2.5	A3	2	S	1042.0	2.10	392.2	-	-	-
5	6	M	2.5	A3	2	S	1042.0	2.73	746.1	-	-	-
4	6	M	2.5	A3	3	S	1563.0	3.94	1154.0	-	-	-
3	6	M	2.5	A3	3	S	1563.0	4.39	1405.1	-	-	-
2	6	M	2.5	A3	2	D	2084.0	5.57	1712.3	-	-	-
1	6	M	2.5	A3	2	D	2084.0	5.66	1644.8	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	0.86	240.6	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.52	621.9	-	-	-
4	4	M	2.5	A3	2	S	1042.0	1.72	756.6	-	-	-
3	4	M	2.5	A3	2	S	1042.0	1.89	905.9	-	-	-
2	4	M	2.5	A3	2	S	1042.0	1.86	858.6	-	-	-
1	4	M	2.5	A3	3	S	1563.0	2.67	1166.3	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	2.5	A3	2	S	1042.0	0.00	0.7	31.26	8.29	3.77
5	2	M	2.5	A3	2	S	1042.0	0.21	174.1	31.26	18.28	1.71
4	2	M	2.5	A3	2	S	1042.0	0.30	261.0	39.08	26.18	1.49
3	2	M	2.5	A3	2	S	1042.0	0.39	376.1	39.08	32.02	1.22
2	2	M	2.5	A3	2	S	1042.0	0.34	311.3	46.89	35.83	1.31
1	2	M	2.5	A3	2	S	1042.0	0.30	266.0	52.10	37.66	1.38

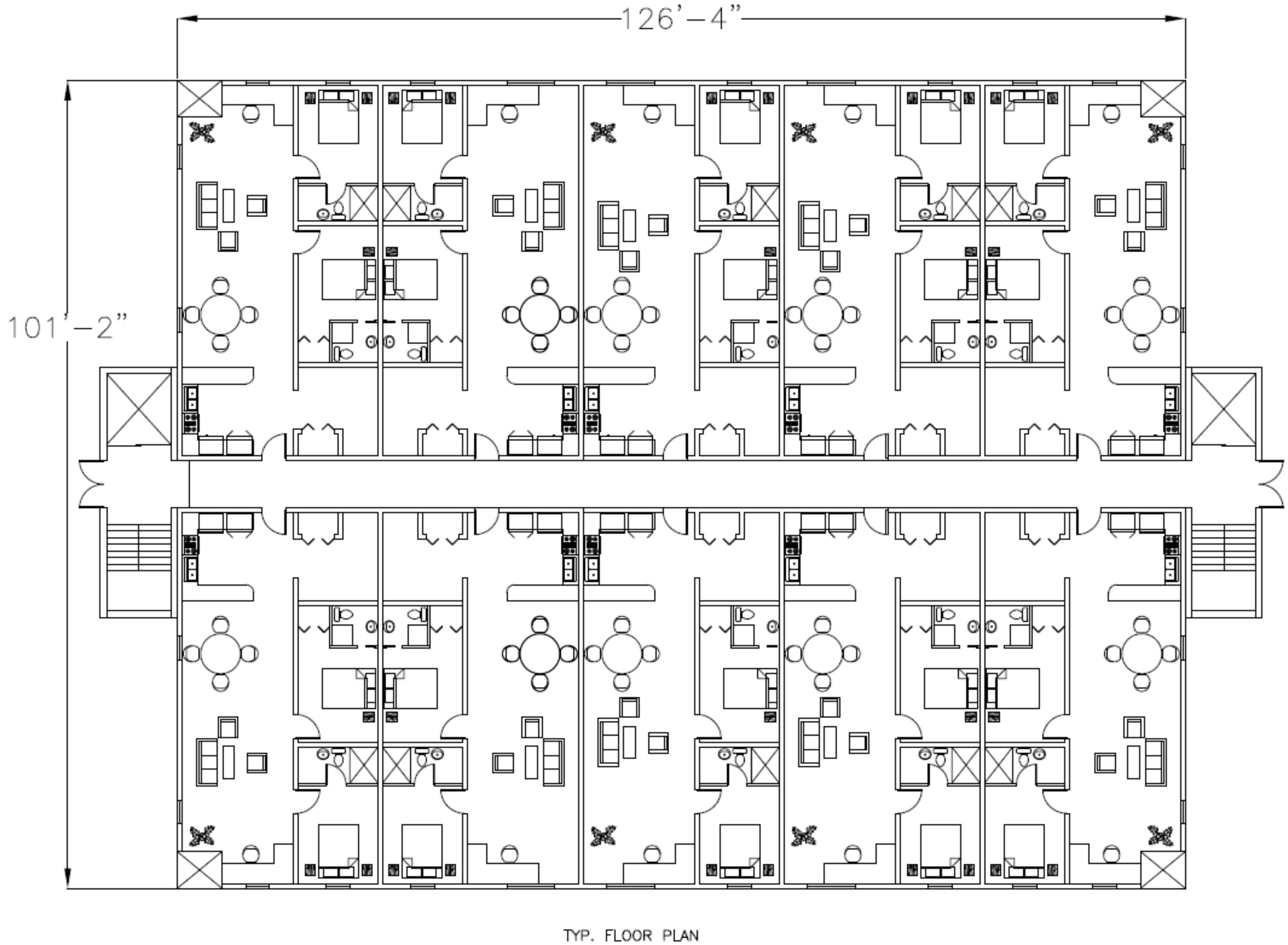


Figure B. 61. Index Bldg. 9 floor plan

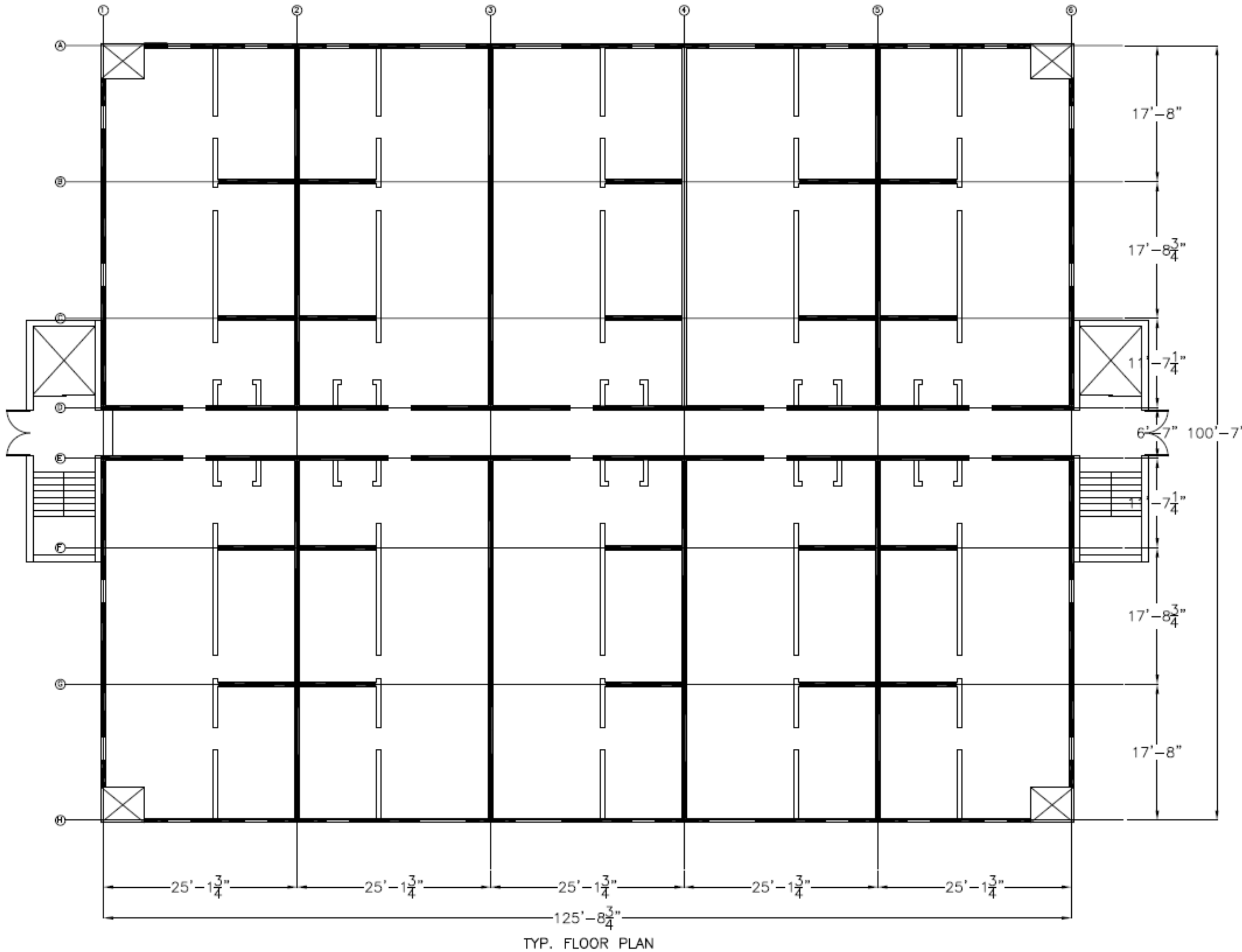


Figure B. 62. Index Bldg. 9 assigned shear walls

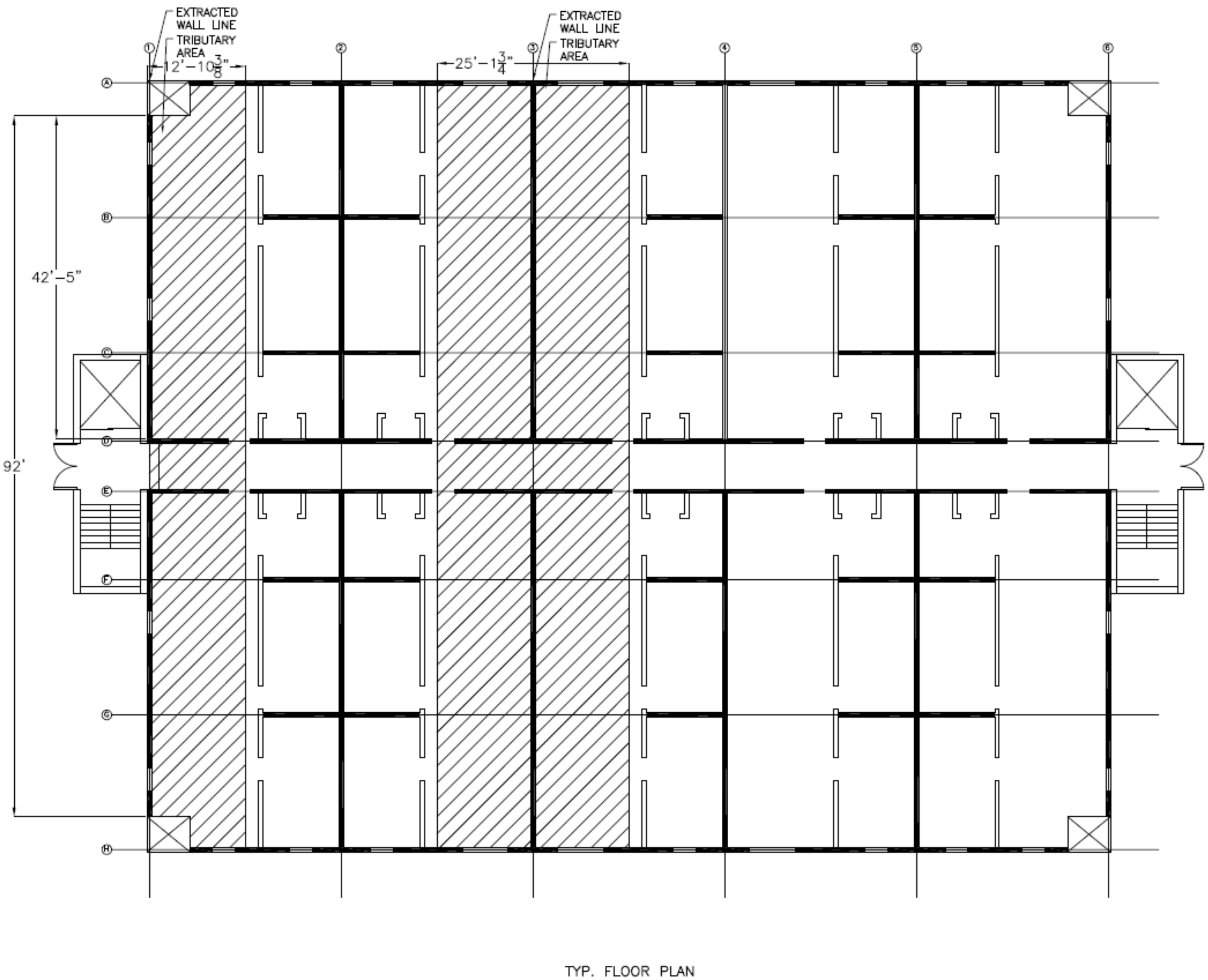


Figure B. 63. Index Bldg. 9 tributary area

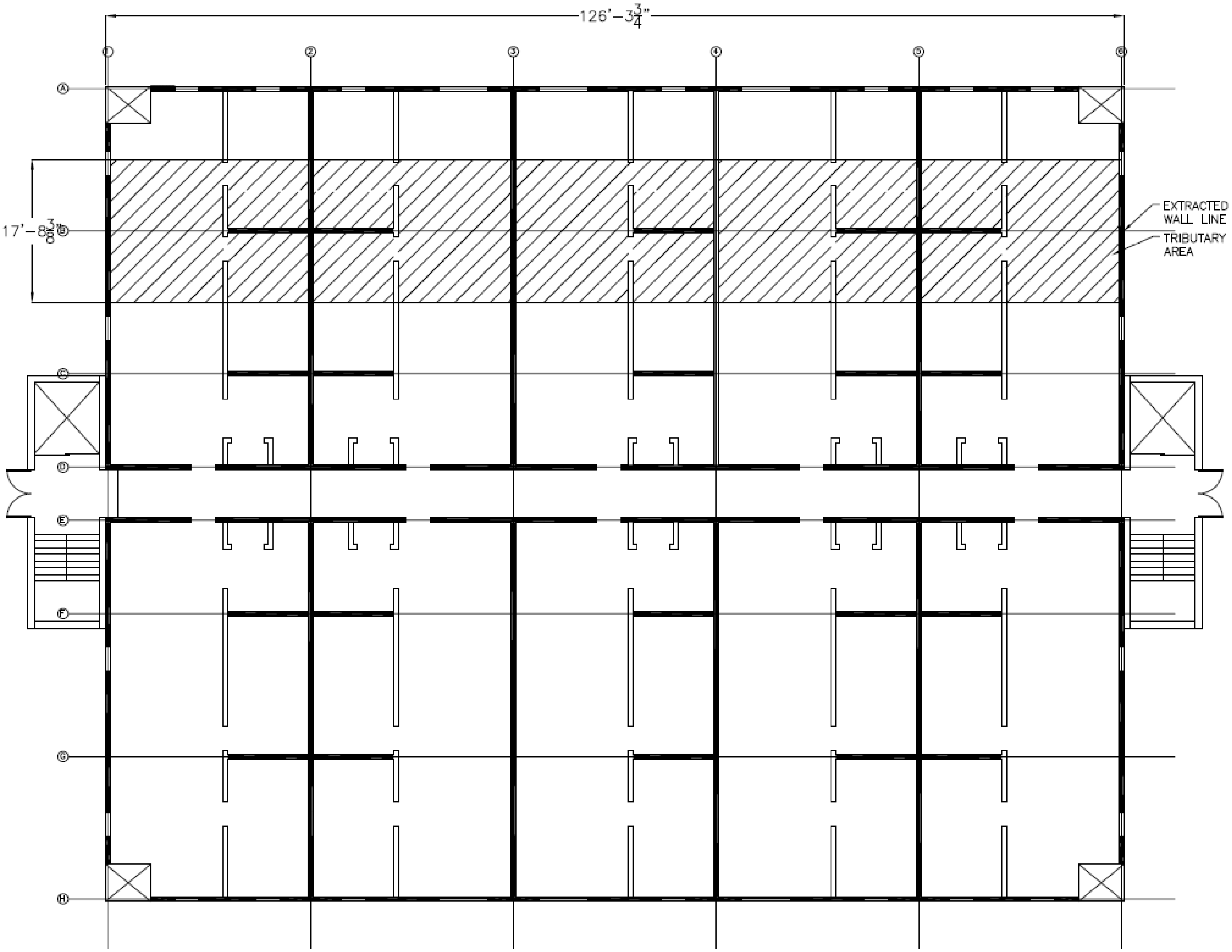


Figure B. 64. Index Bldg. 9 tributary area

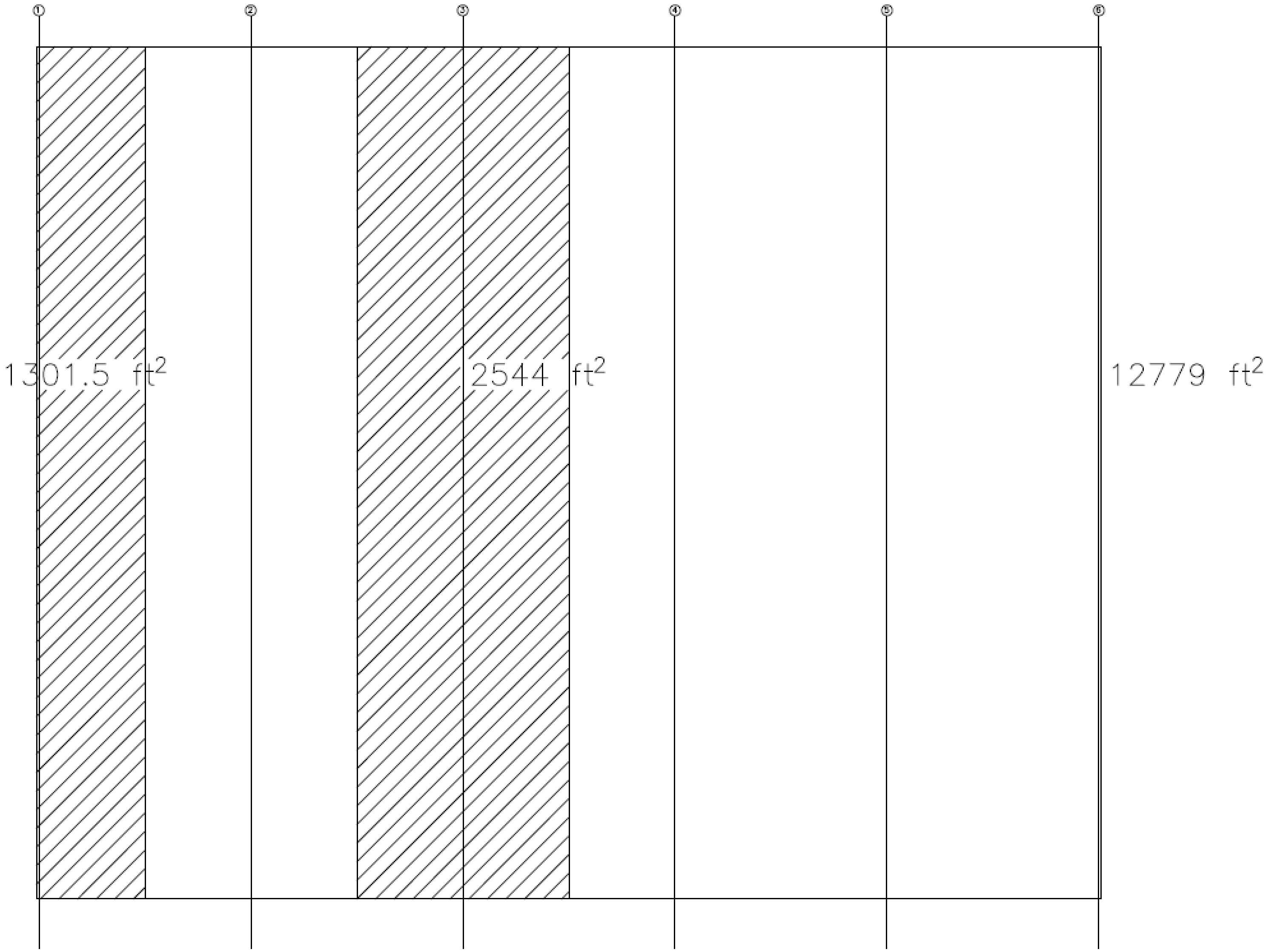


Figure B. 65. Index Bldg. 9 tributary area

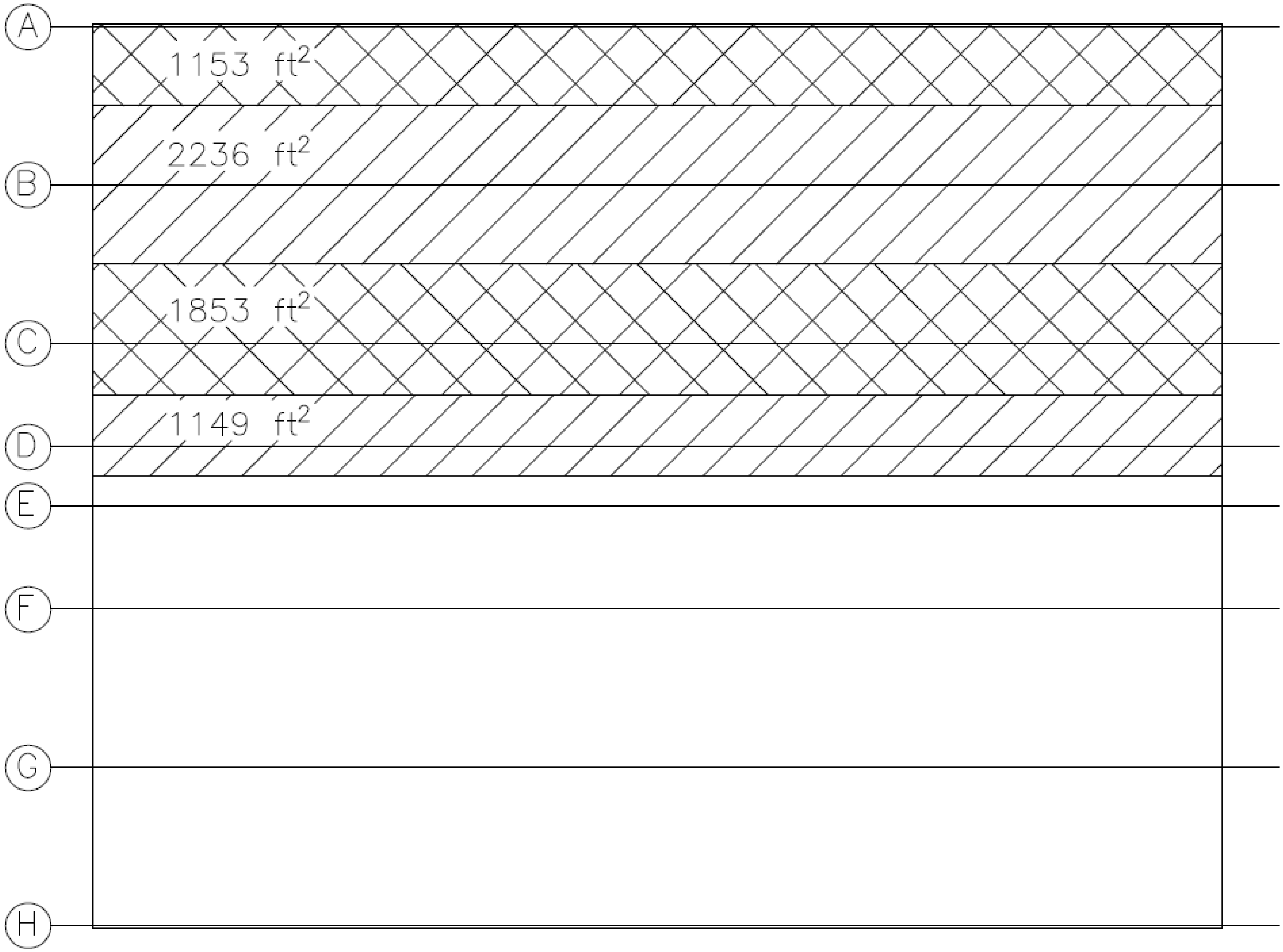


Figure B. 66. Index Bldg. 9 tributary area

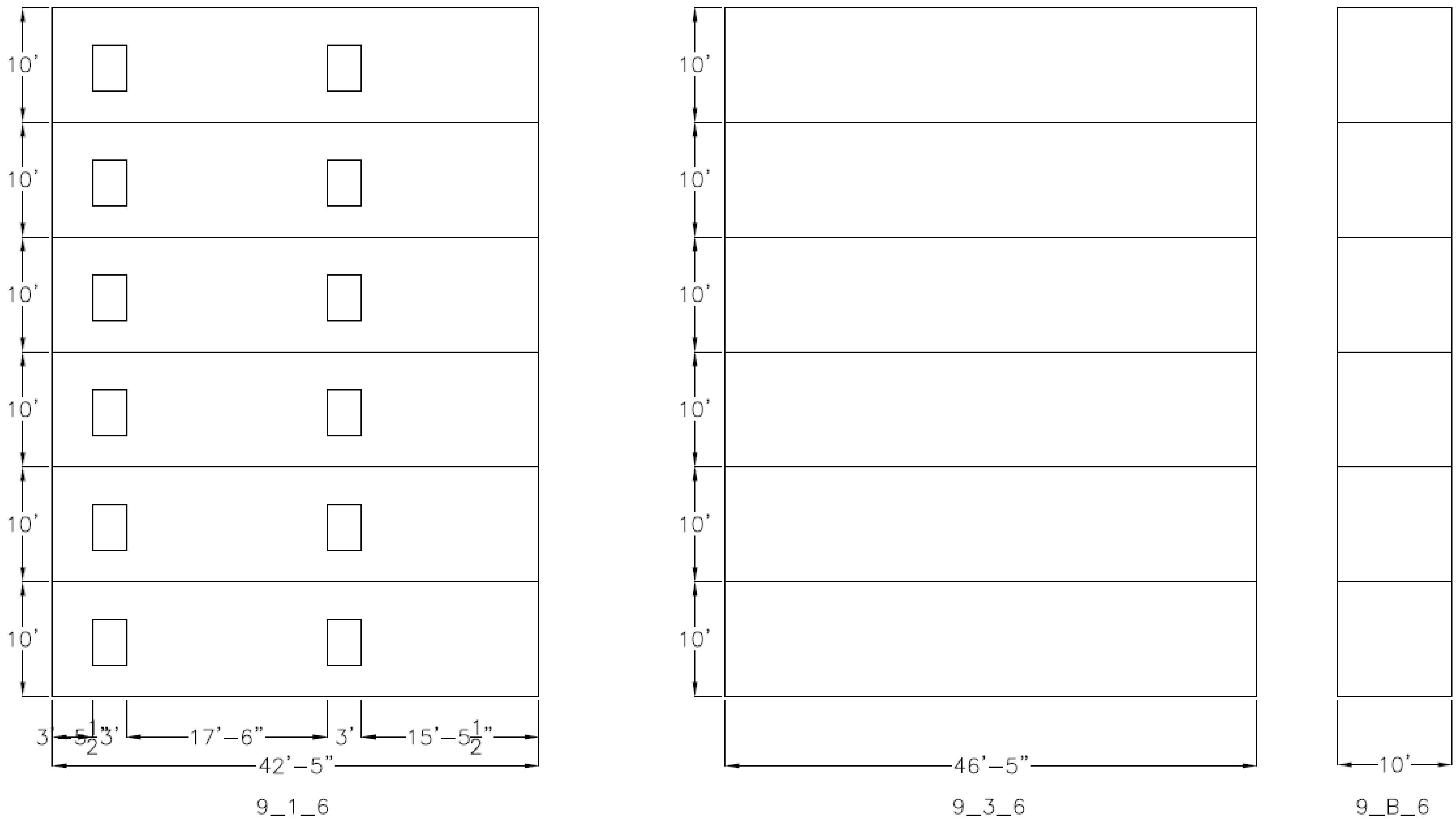


Figure B. 67. Index Bldg. 9 extracted shear wall lines

Table B. 133. Extracted Archetypes, Index Bldg. 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 B. 134. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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 B. 135. Seismic weight summary for Index Bldg. 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				DESIGN SUMMARY	
Total Height	h_n =	60.0	ft	Total base shear	
Total Weight	W=	4,822	k	V =	1,597.63
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			

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 Bldg. 9, 6 story, R=3

INPUT DATA				DESIGN SUMMARY	
Total Height	h_n =	60.0	ft	Total base shear	
Total Weight	W=	5,979	k	V =	1,980.98
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			

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 Bldg. 9, 6 story, R=3

Table B. 136. Tributary load calculation, Index Bldg. 9, 6 story, N-S direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: Cumulative Shear Load (kip)*
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 B. 137. Tributary load calculation, Index Bldg. 9, 6 story, E-W direction, R=3

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: 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 B. 138. Archetype 6 (9_B_6 _1_ LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	A3	3	S	781.5	1.62	644.5	7.82	6.45	1.21
5	2	M	5	A3	4	D	2084.0	4.08	1603.4	20.84	16.03	1.30
4	2	M	5	A3	5	D	2605.0	5.26	2361.7	26.05	23.62	1.10
3	2	M	5	A3	6	D	3126.0	6.65	2922.1	31.26	29.22	1.07
2	2	M	5	A3	7	D	3647.0	7.81	3287.9	36.47	32.88	1.11
1	2	M	5	A3	7	D	3647.0	7.57	3464.3	36.47	34.64	1.05

Table B. 139. Archetype 18 (9_B_6 _1_ LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	2	M	5	A3	3	S	781.5	1.60	630.0	7.82	6.30	1.24
5	2	M	5	A3	3	D	1563.0	3.46	1366.2	15.63	13.66	1.14
4	2	M	5	A3	4	D	2084.0	4.53	1948.3	20.84	19.48	1.07
3	2	M	5	A3	5	D	2605.0	5.62	2378.5	26.05	23.79	1.10
2	2	M	5	A3	6	D	3126.0	6.31	2659.3	31.26	26.59	1.18
1	2	M	5	A3	6	D	3126.0	6.66	2794.8	31.26	27.95	1.12

Table B. 140. Archetype 30 (9_B_6 _1_HR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.54	644.5	10.42	6.45	1.62
5	4	M	2.5	A3	2	D	2084.0	2.84	1603.4	20.84	16.03	1.30
4	4	M	2.5	A3	3	D	3126.0	4.06	2361.7	31.26	23.62	1.32
3	4	M	2.5	A3	3	D	3126.0	4.61	2922.1	31.26	29.22	1.07
2	4	M	2.5	A3	4	D	4168.0	5.75	3287.9	41.68	32.88	1.27
1	4	M	2.5	A3	4	D	4168.0	5.77	3464.3	41.68	34.64	1.20

Table B. 141. Archetype 42 (9_B_6 _1_ HR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.39	630.0	10.42	6.30	1.65
5	4	M	2.5	A3	2	D	2084.0	2.49	1366.2	20.84	13.66	1.53
4	4	M	2.5	A3	2	D	2084.0	2.74	1948.3	20.84	19.48	1.07
3	4	M	2.5	A3	3	D	3126.0	3.71	2378.5	31.26	23.79	1.31
2	4	M	2.5	A3	3	D	3126.0	4.06	2659.3	31.26	26.59	1.18
1	4	M	2.5	A3	3	D	3126.0	4.36	2794.8	31.26	27.95	1.12

Table B. 142. Archetype 78 (9_3_6_2_ LR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	9	M	5	A3	2	S	521.0	10.51	407.4	23.45	18.33	1.28
5	9	M	5	A3	4	S	1042.0	20.05	1013.9	46.89	45.63	1.03
4	9	M	5	A3	3	D	1563.0	29.73	1493.6	70.34	67.21	1.05
3	9	M	5	A3	4	D	2084.0	38.98	1848.0	93.78	83.16	1.13
2	9	M	5	A3	4	D	2084.0	40.80	2079.3	93.78	93.57	1.00
1	9	M	5	A3	5	D	2605.0	41.60	2190.9	117.23	98.59	1.19

Table B. 143. Archetype 90 (9_3_6_2_ LR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	5	A3	2	S	521.0	3.64	448.1	-	-	-
5	4	M	5	A3	4	S	1042.0	7.07	971.7	-	-	-
4	4	M	5	A3	3	D	1563.0	9.93	1386.1	-	-	-
3	4	M	5	A3	4	D	2084.0	12.84	1691.5	-	-	-
2	4	M	5	A3	4	D	2084.0	13.21	1891.3	-	-	-
1	4	M	5	A3	4	D	2084.0	13.65	1987.7	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	5	A3	2	S	521.0	3.64	448.1	20.84	17.92	1.16
5	4	M	5	A3	4	S	1042.0	7.07	971.7	41.68	38.87	1.07
4	4	M	5	A3	3	D	1563.0	9.93	1385.4	62.52	55.43	1.13
3	4	M	5	A3	4	D	2084.0	12.85	1692.0	83.36	67.67	1.23
2	4	M	5	A3	4	D	2084.0	13.21	1891.6	83.36	75.66	1.10
1	4	M	5	A3	4	D	2084.0	13.65	1987.9	83.36	79.51	1.05

Table B. 144. Archetype 102 (9_3_6 _2_ HR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	A3	2	S	1042.0	10.60	407.4	46.89	18.33	2.56
5	18	M	2.5	A3	2	S	1042.0	12.85	1013.9	46.89	45.63	1.03
4	18	M	2.5	A3	2	D	2084.0	22.12	1493.6	93.78	67.21	1.40
3	18	M	2.5	A3	2	D	2084.0	24.07	1848.0	93.78	83.16	1.13
2	18	M	2.5	A3	2	D	2084.0	25.47	2079.3	93.78	93.57	1.00
1	18	M	2.5	A3	3	D	3126.0	32.97	2190.9	140.67	98.59	1.43

Table B. 145. Archetype 114 (9_3_6 _2_ HR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	A3	2	S	1042.0	10.55	398.3	46.89	17.92	2.62
5	18	M	2.5	A3	2	S	1042.0	12.61	863.7	46.89	38.87	1.21
4	18	M	2.5	A3	2	D	2084.0	21.54	1231.8	93.78	55.43	1.69
3	18	M	2.5	A3	2	D	2084.0	23.47	1503.8	93.78	67.67	1.39
2	18	M	2.5	A3	2	D	2084.0	24.86	1681.3	93.78	75.66	1.24
1	18	M	2.5	A3	2	D	2084.0	24.95	1767.0	93.78	79.51	1.18

Table B. 146. Archetype 126 (9_3_6_2_ MR_HG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.11	335.6	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.30	411.3	-	-	-
4	4	M	2.5	A3	2	S	1042.0	1.47	517.0	-	-	-
3	4	M	2.5	A3	2	S	1042.0	1.53	540.6	-	-	-
2	4	M	2.5	A3	2	S	1042.0	1.50	511.0	-	-	-
1	4	M	2.5	A3	2	S	1042.0	1.59	553.4	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	2	S	521.0	2.49	499.3	-	-	-
5	3	M	5	A3	3	D	1563.0	6.57	1383.8	-	-	-
4	3	M	5	A3	4	D	2084.0	8.83	2068.0	-	-	-
3	3	M	5	A3	5	D	2605.0	11.01	2591.8	-	-	-
2	3	M	5	A3	6	D	3126.0	13.01	2948.7	-	-	-
1	3	M	5	A3	6	D	3126.0	13.38	3101.9	-	-	-

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	2	S	521.0	2.49	499.3	26.05	18.33	1.42
5	3	M	5	A3	3	D	1563.0	6.57	1383.8	57.31	45.63	1.26
4	3	M	5	A3	4	D	2084.0	8.83	2068.0	72.94	67.21	1.09
3	3	M	5	A3	5	D	2605.0	11.01	2591.8	88.57	83.16	1.07
2	3	M	5	A3	6	D	3126.0	13.01	2948.7	104.20	93.57	1.11
1	3	M	5	A3	6	D	3126.0	13.38	3101.9	104.20	98.59	1.06

Table B. 147. Archetype 138 (9_3_6_2_ MR_LG_ DX_LP) design, R=3

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	0.67	150.9	-	-	-
5	4	M	2.5	A3	2	S	1042.0	1.33	410.5	-	-	-
4	4	M	2.5	A3	2	S	1042.0	1.31	409.5	-	-	-
3	4	M	2.5	A3	2	S	1042.0	1.51	548.1	-	-	-
2	4	M	2.5	A3	2	S	1042.0	1.43	465.3	-	-	-
1	4	M	2.5	A3	2	S	1042.0	1.49	510.0	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	4	S	1042.0	3.66	547.1	-	-	-
5	3	M	5	A3	5	S	1302.5	5.63	1158.8	-	-	-
4	3	M	5	A3	4	D	2084.0	8.24	1711.2	-	-	-
3	3	M	5	A3	4	D	2084.0	8.57	2072.7	-	-	-
2	3	M	5	A3	5	D	2605.0	10.88	2367.2	-	-	-
1	3	M	5	A3	5	D	2605.0	10.90	2480.6	-	-	-
No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	3	M	5	A3	4	S	1042.0	3.66	547.1	41.68	17.92	2.33
5	3	M	5	A3	5	S	1302.5	5.63	1158.7	49.50	38.87	1.27
4	3	M	5	A3	4	D	2084.0	8.24	1711.2	72.94	55.43	1.32
3	3	M	5	A3	4	D	2084.0	8.57	2073.2	72.94	67.67	1.08
2	3	M	5	A3	5	D	2605.0	10.88	2366.5	88.57	75.66	1.17
1	3	M	5	A3	5	D	2605.0	10.90	2480.3	88.57	79.51	1.11

INPUT DATA				DESIGN SUMMARY		
Total Height	h_n=	60.0	ft	Total base shear		
Total Weight	W=	4,822	k	V	=	1,198.22
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				

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 Bldg. 9, 6 story, R=4

INPUT DATA				DESIGN SUMMARY		
Total Height	h_n=	60.0	ft	Total base shear		
Total Weight	W=	5,979	k	V	=	1,485.73
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				

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 Bldg. 9, 6 story, R=4

Table B. 148. Tributary load calculation, Index Bldg. 9, 6 story, N-S direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: 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 B. 149. Tributary load calculation, Index Bldg. 9, 6 story, E-W direction, R=4

	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Low gravity: Story shear (kip)	Low gravity: Cumulative Shear Load (kip)*	High gravity: Story shear (kip)	High gravity: 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 B. 150. Archetype 30 (9_B_6 _1_HR_HG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.36	483.3	10.42	4.83	2.16
5	4	M	2.5	A3	3	S	1563.0	2.29	1202.8	15.63	12.03	1.30
4	4	M	2.5	A3	2	D	2084.0	3.18	1771.8	20.84	17.72	1.18
3	4	M	2.5	A3	3	D	3126.0	4.18	2192.2	31.26	21.92	1.43
2	4	M	2.5	A3	3	D	3126.0	4.65	2466.7	31.26	24.67	1.27
1	4	M	2.5	A3	3	D	3126.0	4.73	2599.0	31.26	25.99	1.20

Table B. 151. Archetype 42 (9_B_6 _1_ HR_LG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	4	M	2.5	A3	2	S	1042.0	1.24	472.5	10.42	4.73	2.21
5	4	M	2.5	A3	2	S	1042.0	1.69	1024.7	10.42	10.25	1.02
4	4	M	2.5	A3	3	S	1563.0	2.18	1461.3	15.63	14.61	1.07
3	4	M	2.5	A3	2	D	2084.0	2.75	1783.9	20.84	17.84	1.17
2	4	M	2.5	A3	2	D	2084.0	3.02	1994.5	20.84	19.95	1.04
1	4	M	2.5	A3	3	D	3126.0	3.87	2096.1	31.26	20.96	1.49

Table B. 152. Archetype 102 (9_3_6 _2_ HR_HG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	A	2	S	1042.0	9.91	305.6	46.89	13.75	3.41
5	18	M	2.5	A	2	S	1042.0	12.45	760.5	46.89	34.22	1.37
4	18	M	2.5	A	3	S	1563.0	18.02	1120.2	70.34	50.41	1.40
3	18	M	2.5	A	3	S	1563.0	19.27	1386.0	70.34	62.37	1.13
2	18	M	2.5	A	3	S	1563.0	20.22	1559.5	70.34	70.18	1.00
1	18	M	2.5	A	2	D	2084.0	24.78	1643.2	93.78	73.94	1.27

Table B. 153. Archetype 114 (9_3_6 _2_ HR_LG_ DX_LP) design, R=4

No. of Stories	No. of Panels	Configuration	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)	Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
6	18	M	2.5	A	2	S	1042.0	9.86	298.8	46.89	13.44	3.49
5	18	M	2.5	A	2	S	1042.0	12.15	647.8	46.89	29.15	1.61
4	18	M	2.5	A	2	S	1042.0	13.35	923.9	46.89	41.57	1.13
3	18	M	2.5	A	3	S	1563.0	18.76	1127.9	70.34	50.75	1.39
2	18	M	2.5	A	3	S	1563.0	19.70	1261.0	70.34	56.75	1.24
1	18	M	2.5	A	3	S	1563.0	19.83	1325.2	70.34	59.64	1.18

APPENDIX C: TEST RESULTS

Test 1, 2

Test #	Grade	Height (ft)	Length (ft)	# 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

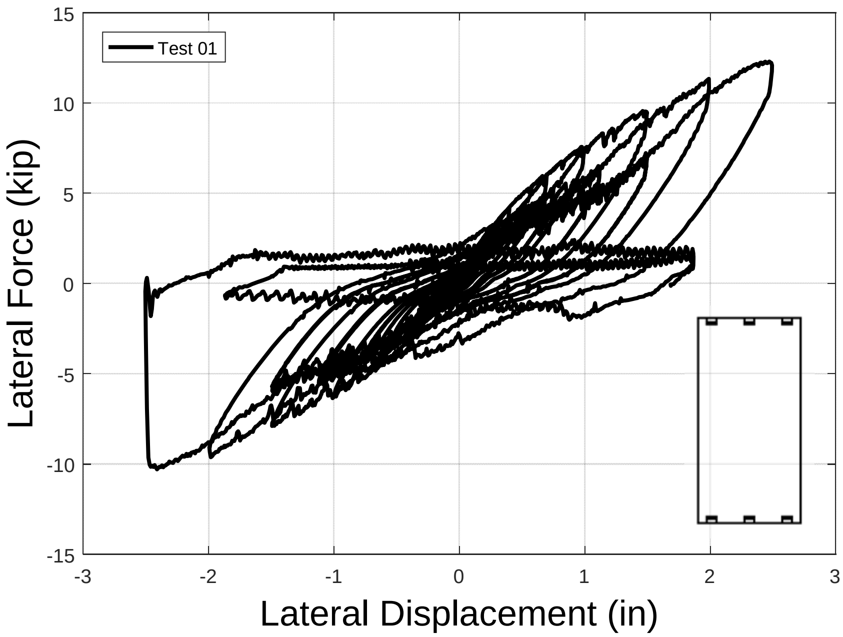


Figure C. 1. Test 01 hysteresis

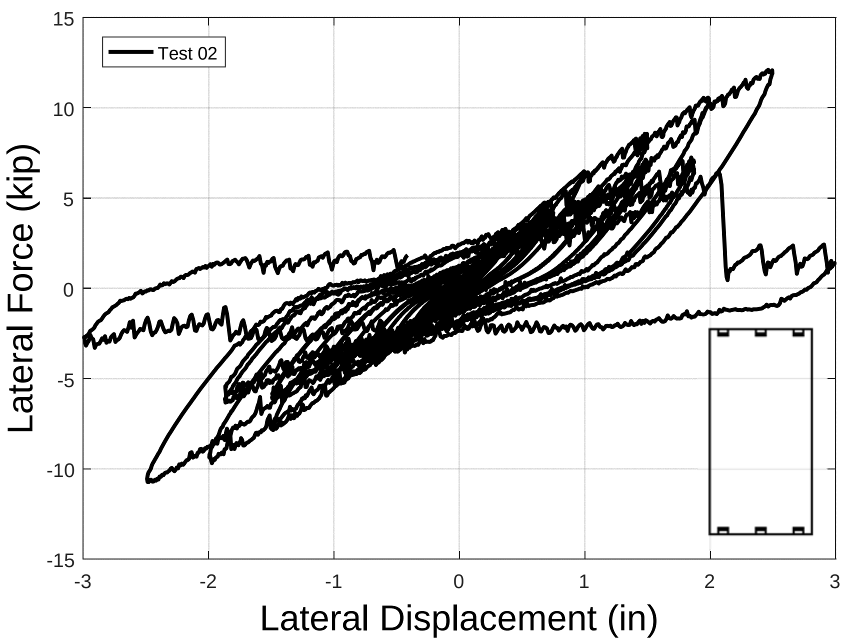


Figure C. 2. Test 02 hysteresis

Test 03, 04

Test #	Grade	Height (ft)	Length (ft)	# 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

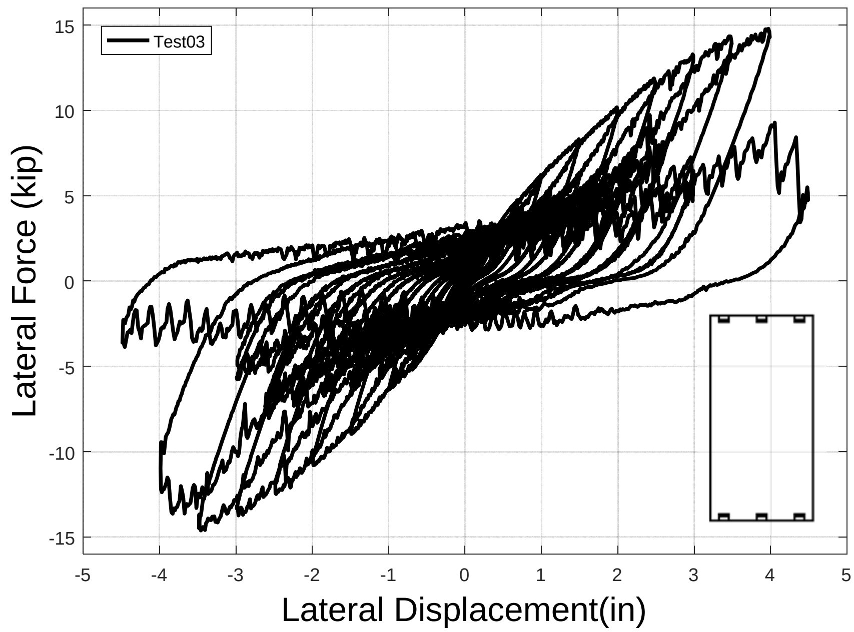


Figure C. 3. Test 03 hysteresis

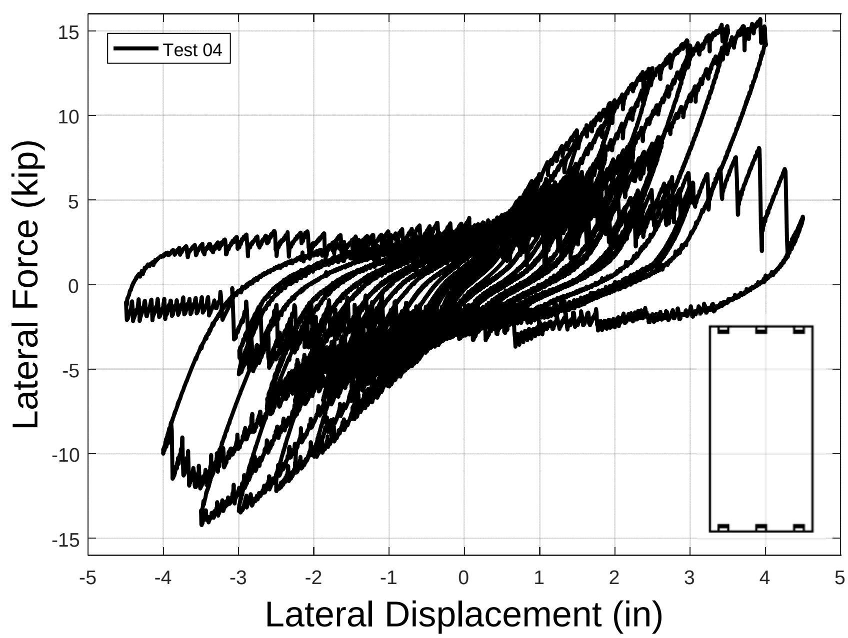


Figure C. 4. Test 04 hysteresis

Test 05, 06

Test #	Grade	Height (ft)	Length (ft)	# 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

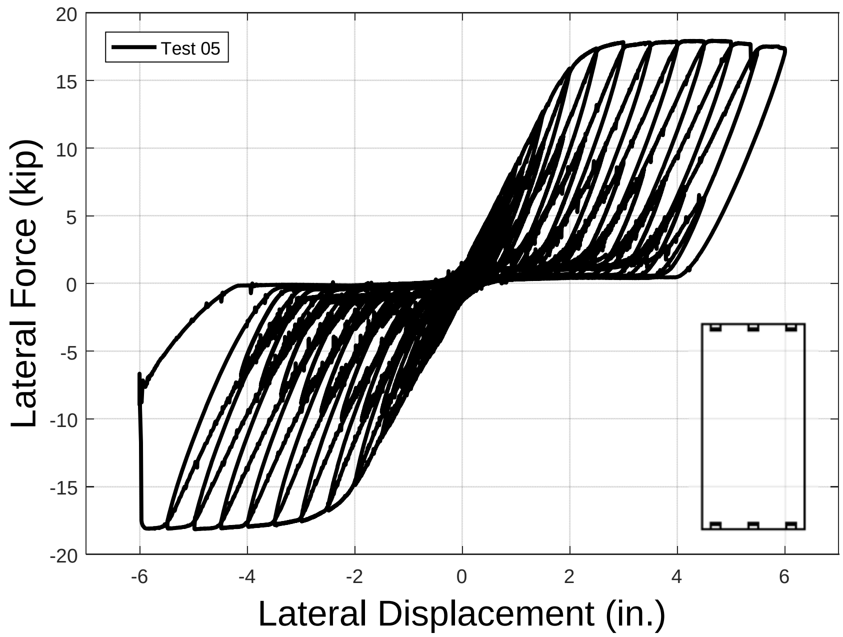


Figure C. 5. Test 05 hysteresis

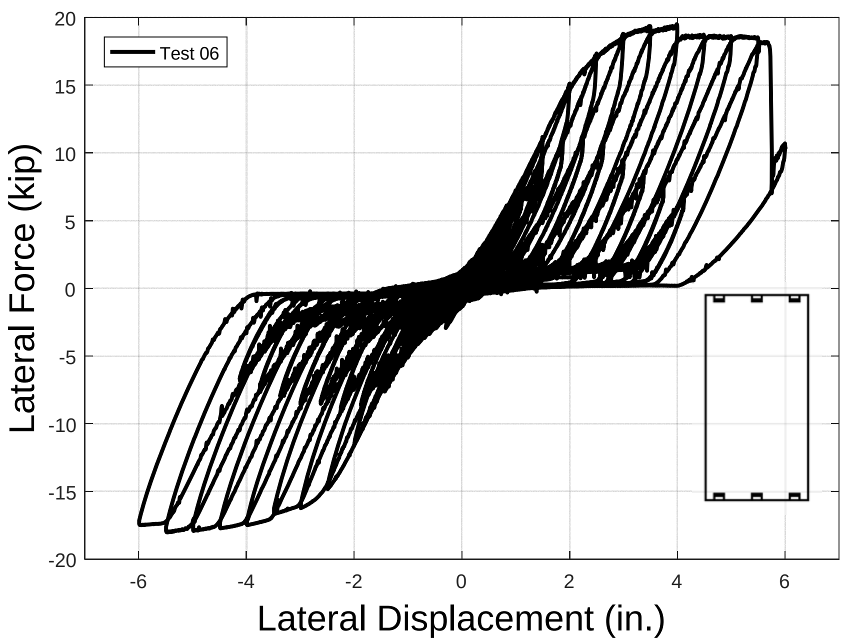


Figure C. 6. Test 06 hysteresis

Test 09, 10

Test #	Grade	Height (ft)	Length (ft)	# 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	-

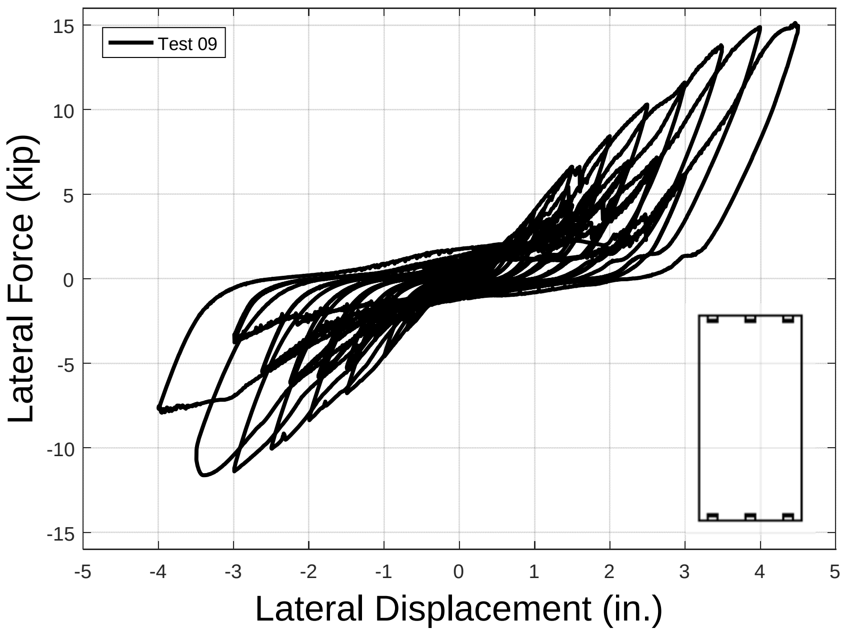


Figure C. 7. Test 09 hysteresis

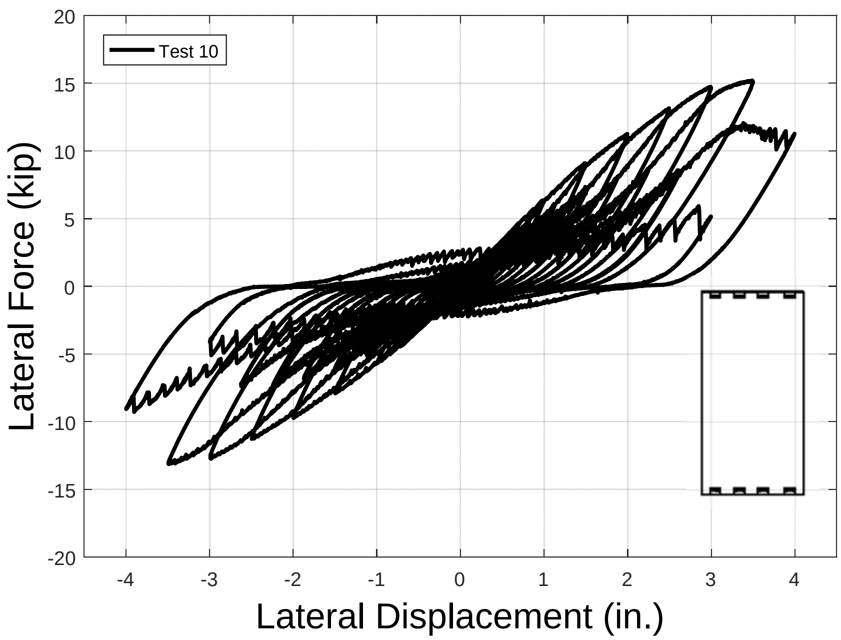


Figure C. 8. Test 10 hysteresis

Test 11, 13

Test #	Grade	Height (ft)	Length (ft)	# 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	-

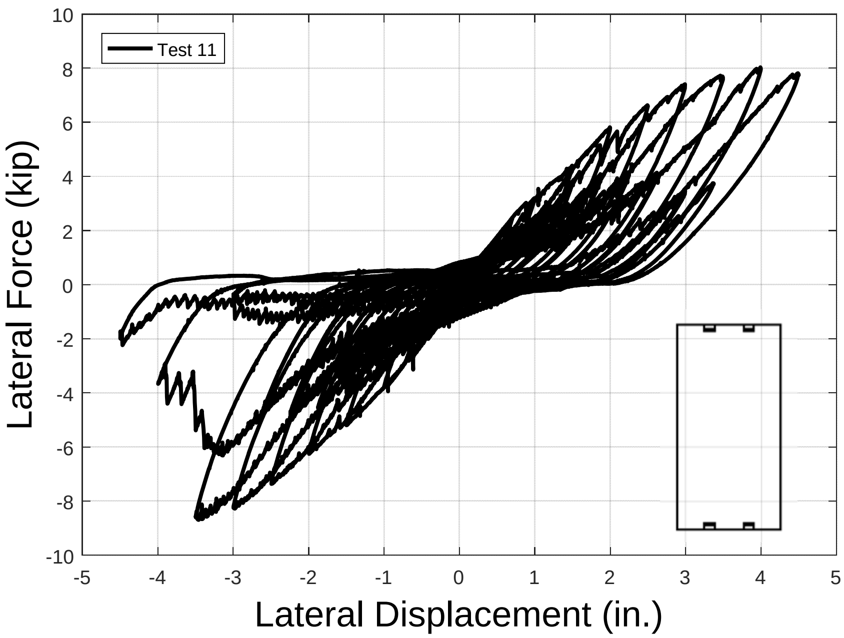


Figure C. 9. Test 11 hysteresis

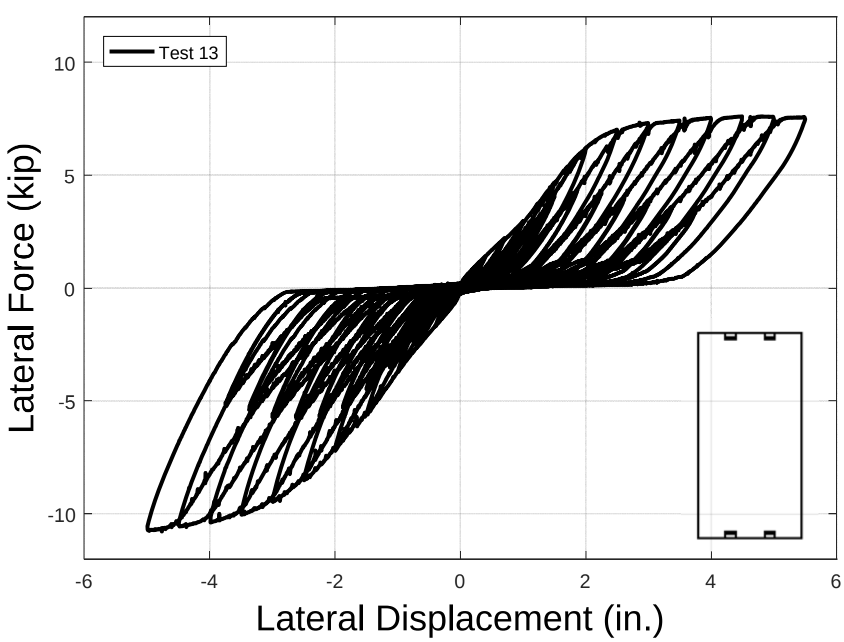


Figure C. 10. Test 13 hysteresis

Test 14, 15

Test #	Grade	Height (ft)	Length (ft)	# 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	-

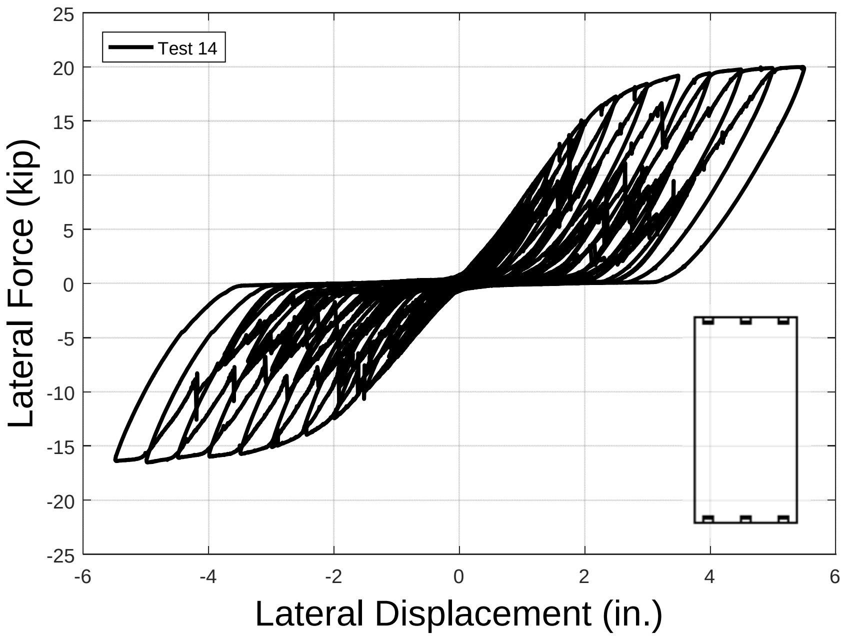


Figure C. 11. Test 14 hysteresis

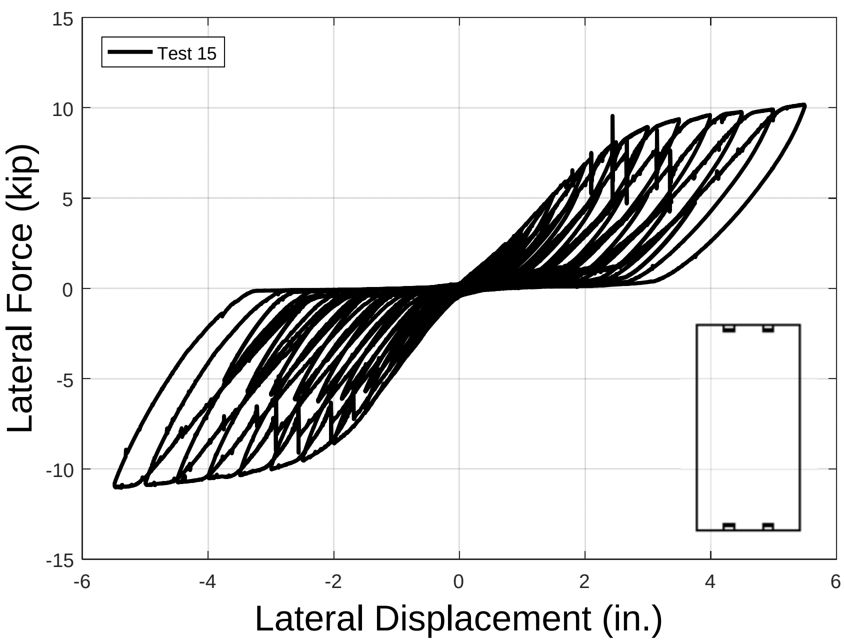


Figure C. 12. Test 15 hysteresis

Test 17, 18

Test #	Grade	Height (ft)	Length (ft)	# 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	-

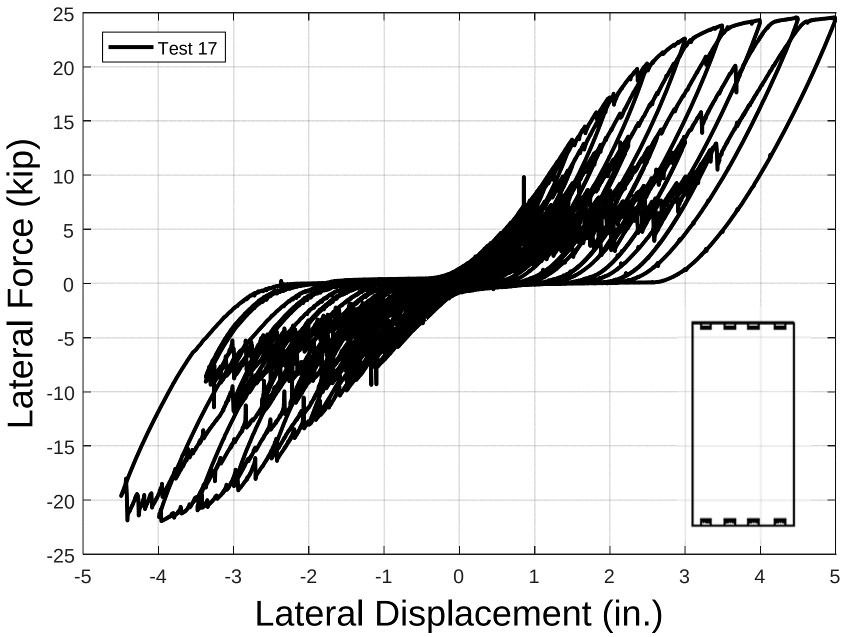


Figure C. 13. Test 17 hysteresis

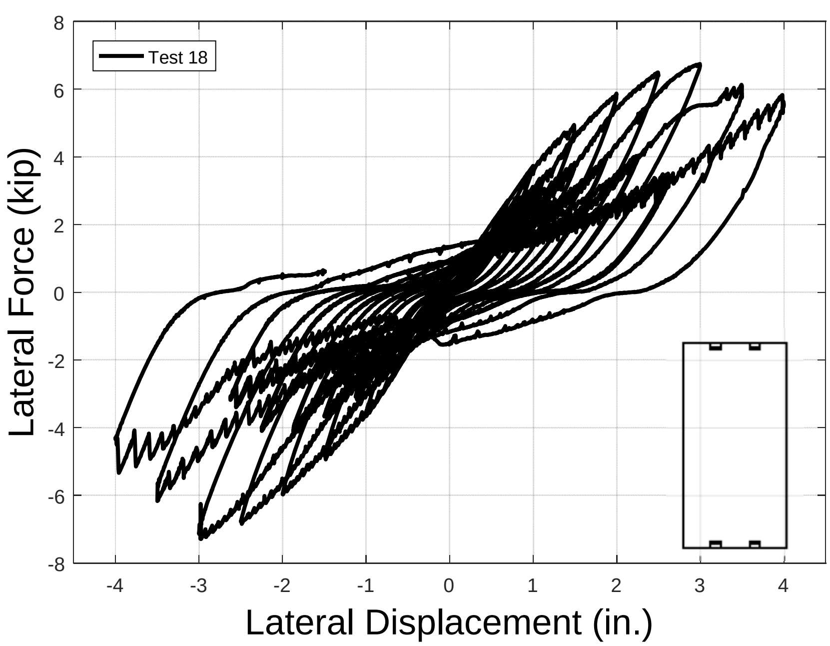


Figure C. 14. Test 18 hysteresis

Test 19, 20

Test #	Grade	Height (ft)	Length (ft)	# 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	-

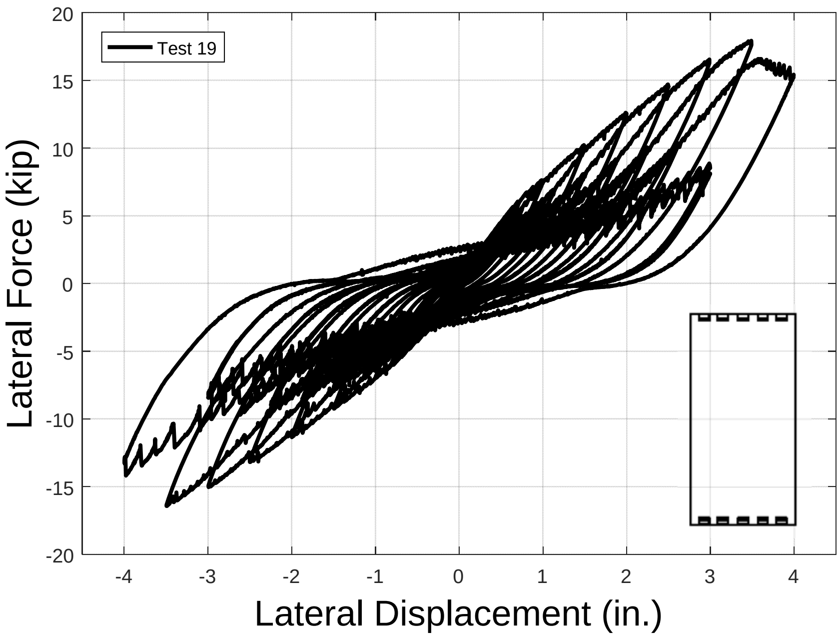


Figure C. 15. Test 19 hysteresis

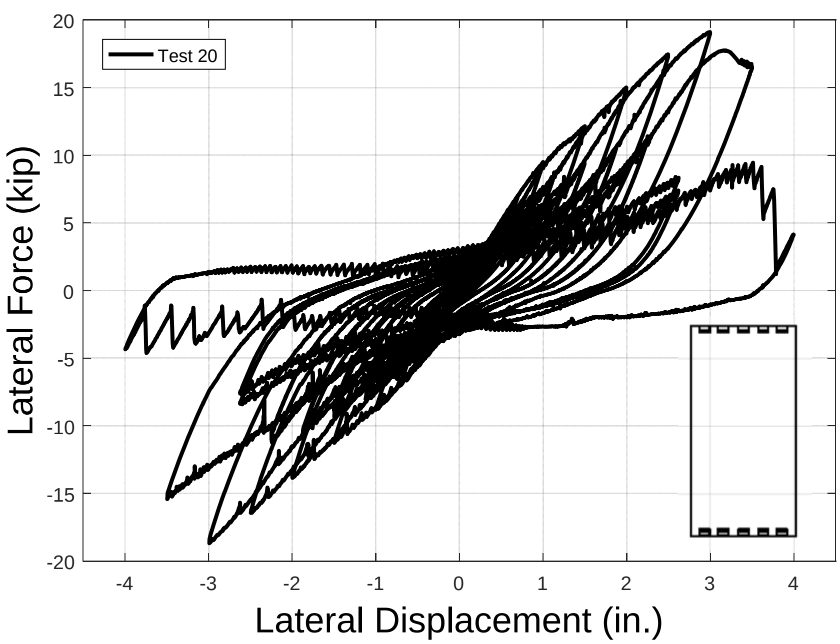


Figure C. 16. Test 20 hysteresis

Test 21, 23

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)
21	V2	8	2	3	3.9	2	A3	-
22	V2	8	8	3	3.9	4	A3	-

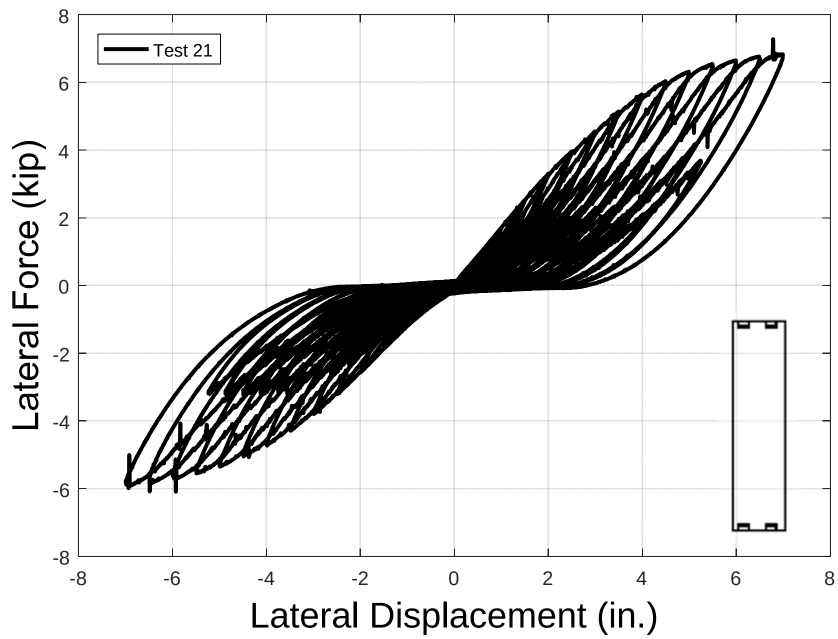


Figure C. 17. Test 21 hysteresis

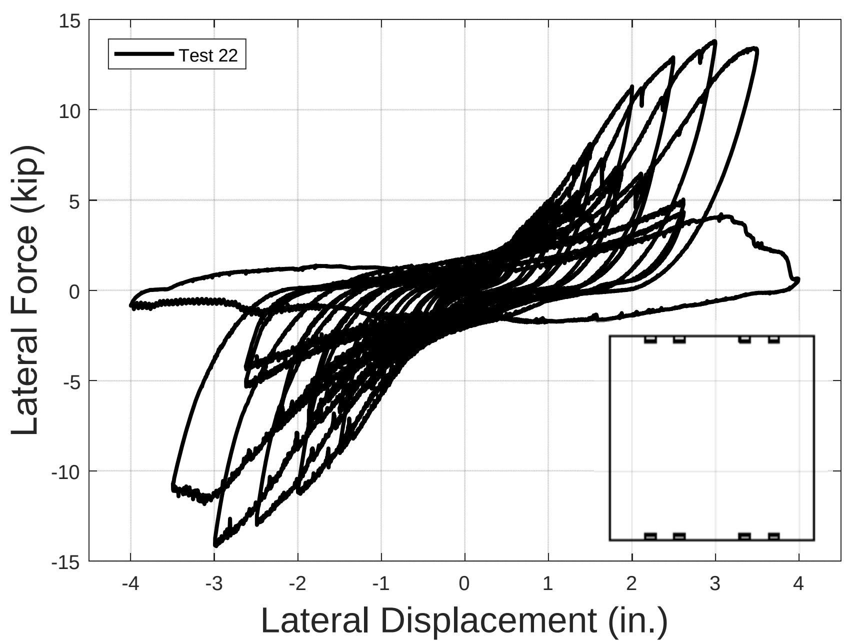


Figure C. 18. Test 22 hysteresis

Test 23, 24

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)
23	V2	8	2 (2)	5	6.65	4	A3	-
24	E1	8	4	5	6.89	2	B3	-

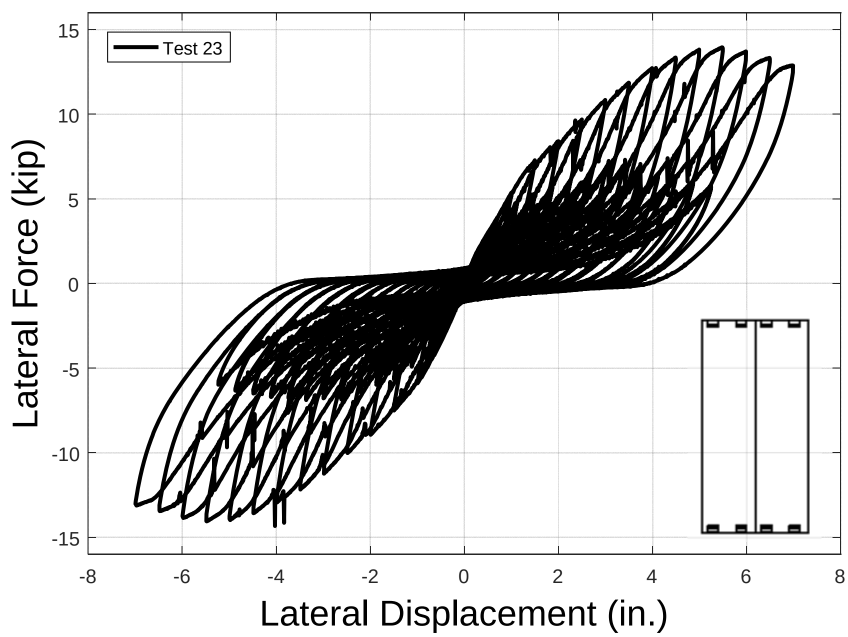


Figure C. 19. Test 23 hysteresis

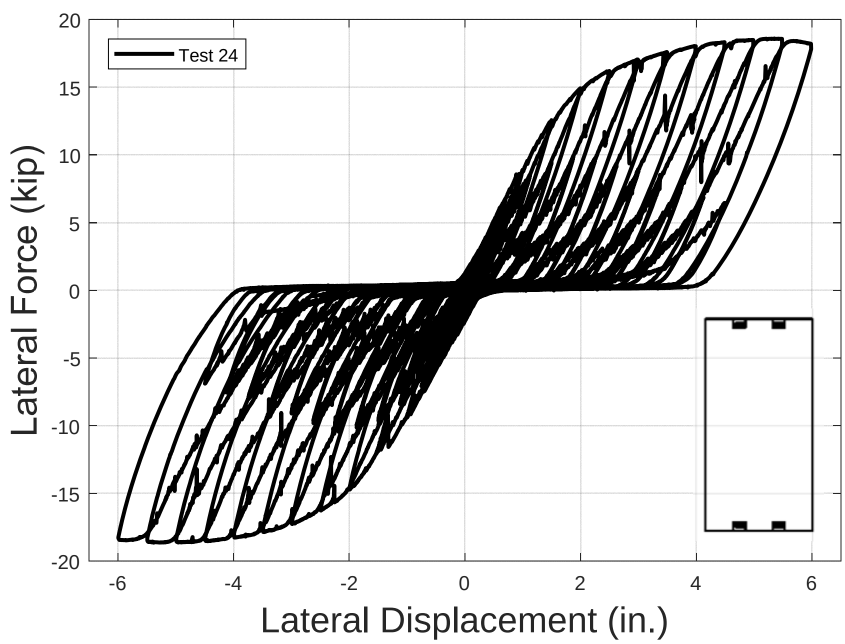


Figure C. 20. Test 24 hysteresis

Test 25, 26

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)
25	E1	8	4	5	6.65	3	B3	-
26	V2	8	4 (2)	5	6.65	8	A3	-

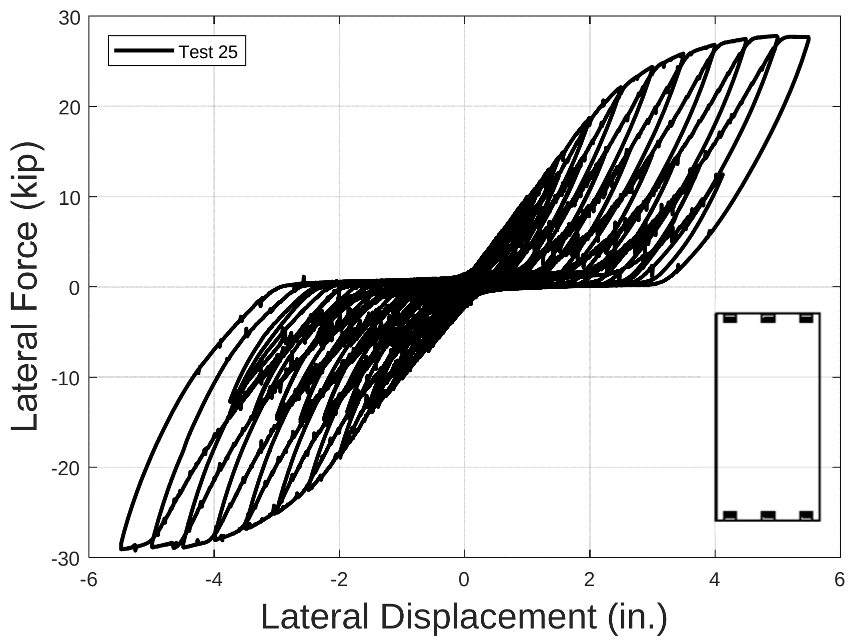


Figure C. 21. Test 25 hysteresis

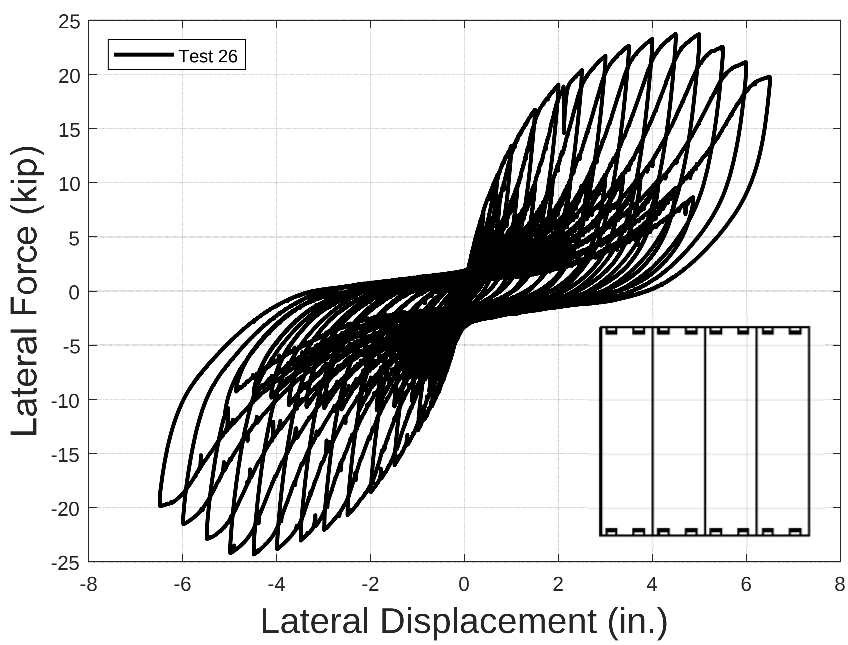


Figure C. 22. Test 26 hysteresis

Test 27, 28

Test #	Grade	Height (ft)	Length (ft)	# Plys	Thickness (in.)	No. connectors	Connector type	Gravity Load (kip/ft)
27	E1	8	4	5	6.89	3	B3(3/16 in.)	-
28	E1	8	4	5	6.89	2	B3 (10 gauge)	-

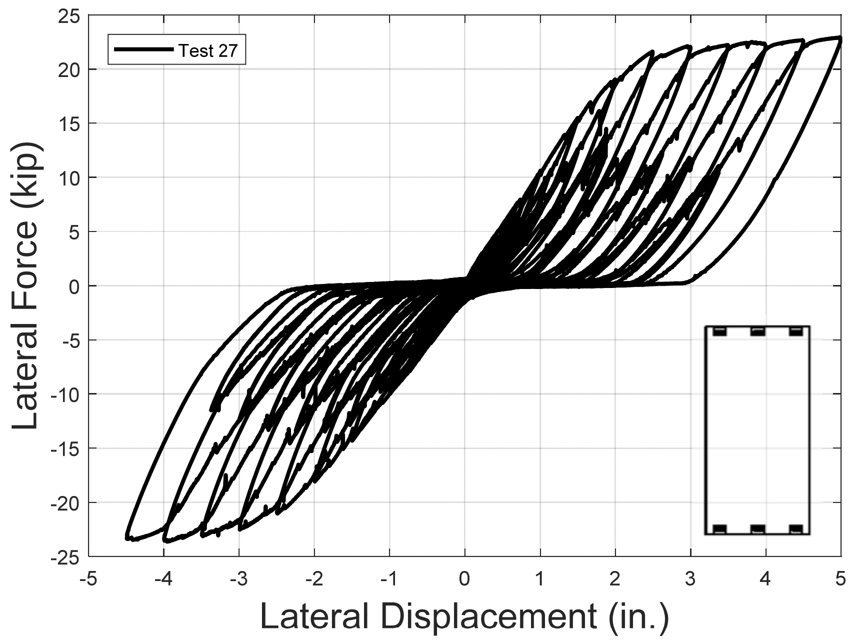


Figure C. 23. Test 27 hysteresis

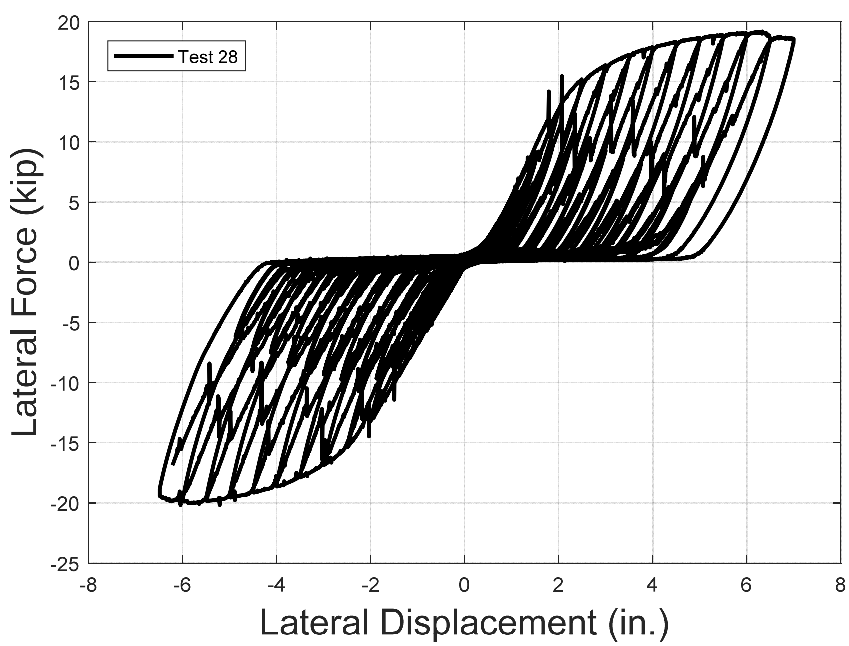


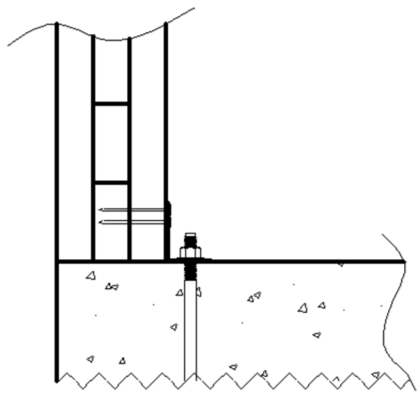
Figure C. 24. Test 28 hysteresis

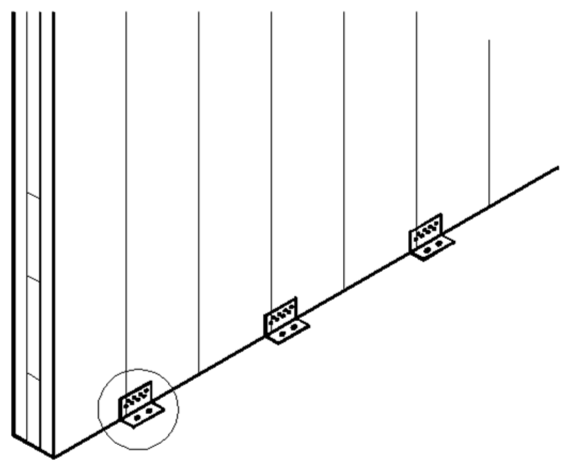
APPENDIX D: DETAILING

Example typical details provided in part Table 12 are covered by the proposed design methodology while other details can be utilized if shown to be compatible with the design methodology.

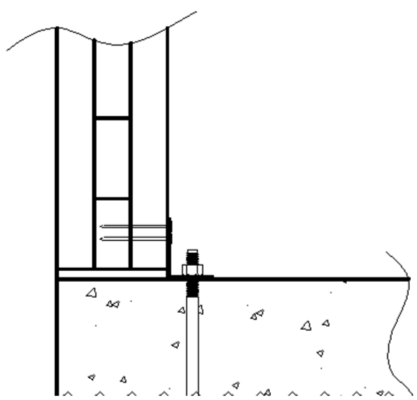
Table D. 1. Typical Connection Details

Base Connections Detail 1

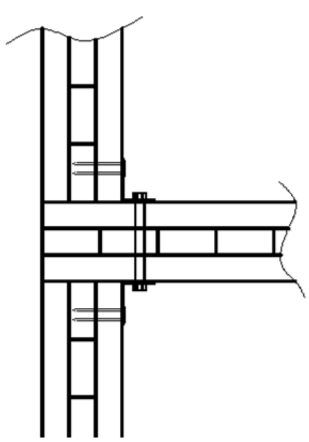


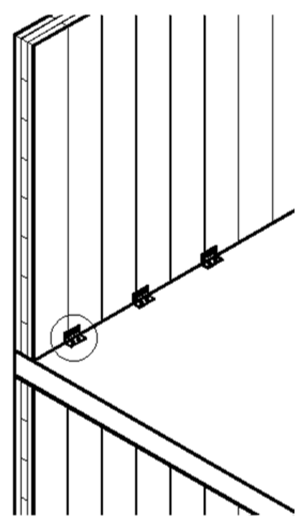


Base Connection Detail 2

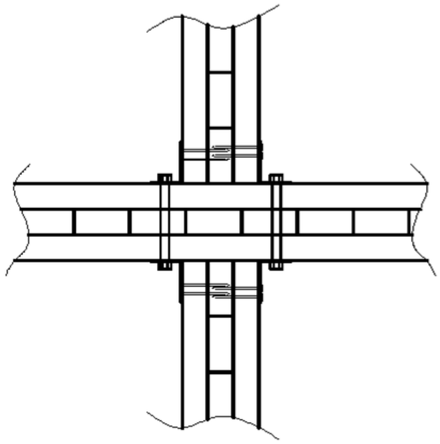


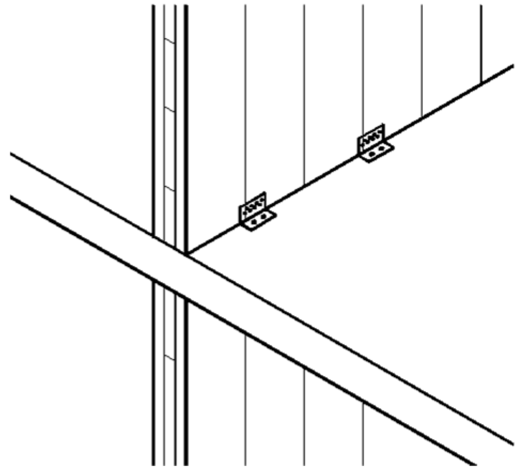
Exterior Wall to Floor Detail 1



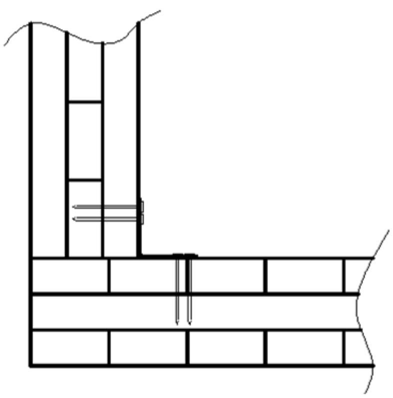


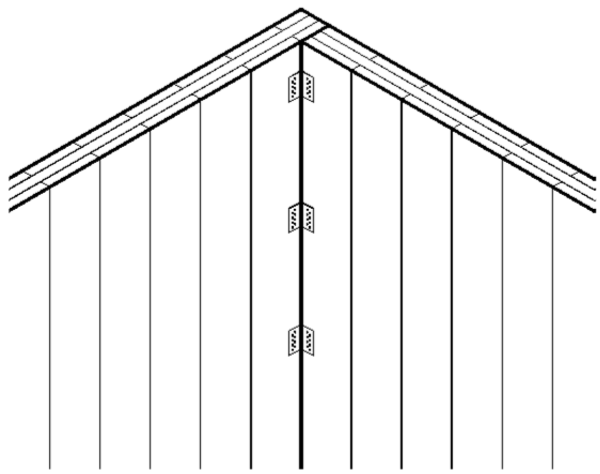
Interior Wall to Floor Detail 1



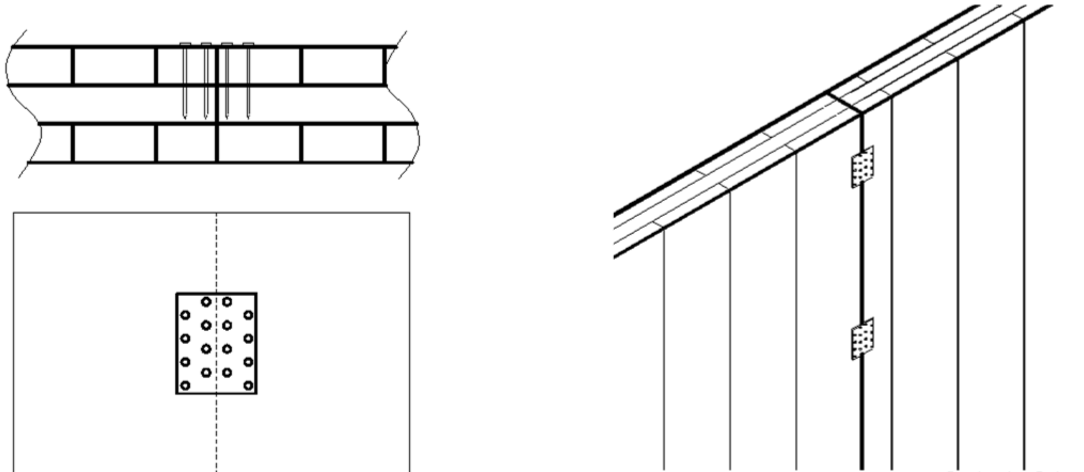


Corner Wall Joint

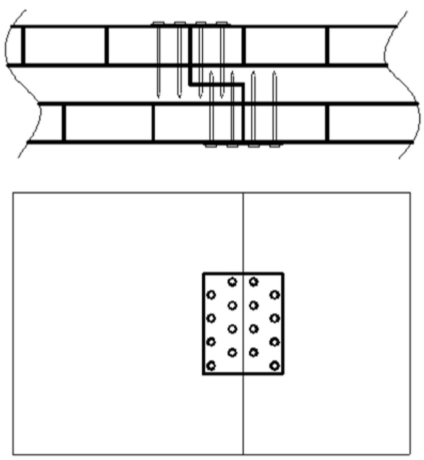


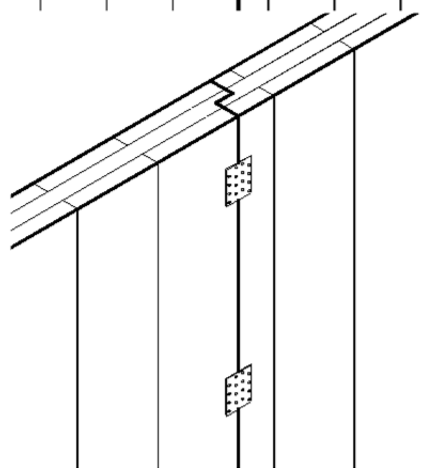


Inter-panel Connection Detail 1



Inter-panel Connection Detail 2





Multi-story overturning restraint

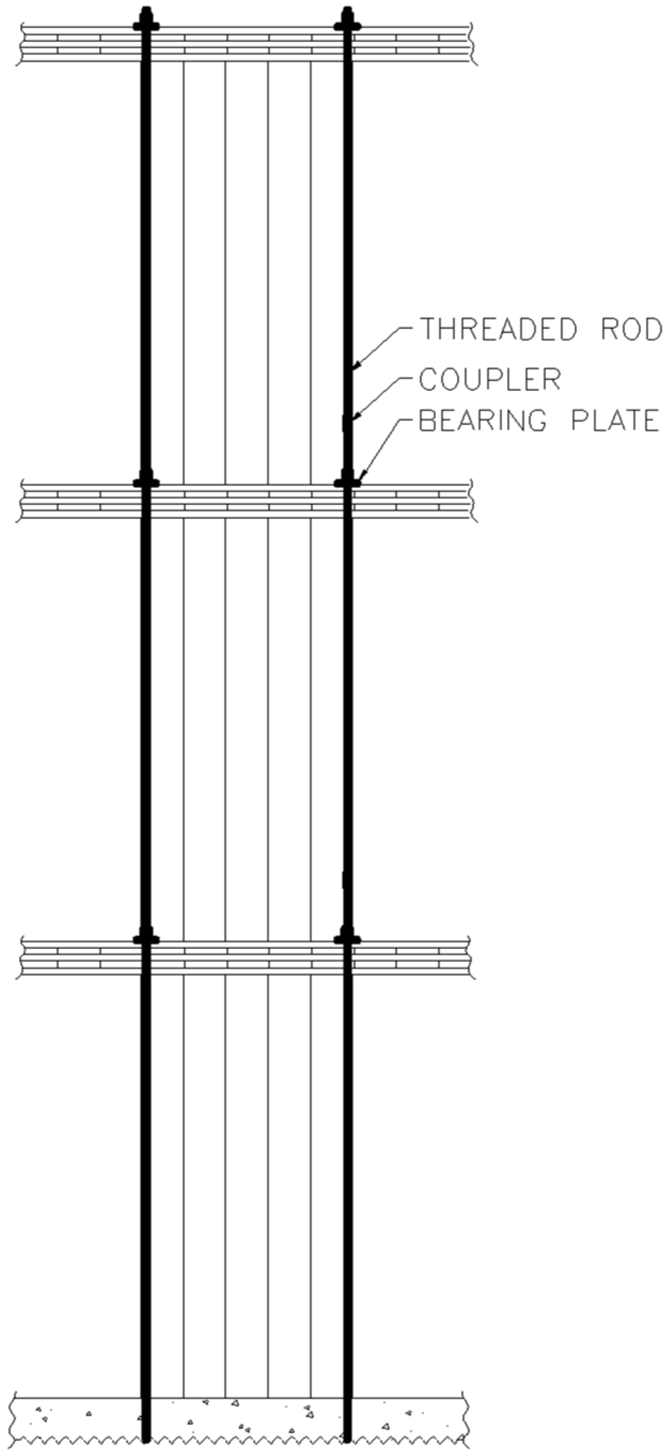


Figure D. 1. Tie-down detail

APPENDIX E: CUREE PARAMETERS USED IN MODELING

Table E. 1. Hysteretic parameters used for nonlinear analysis

Configuration														
Panel length (ft)	No. of panels	Connector Type	No. of connector 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	2250	7250	650	0.075	-0.125	0.825	0.05	8.75	0.75	1.05
2.5	1	A3	2	D	4500	14500	1300	0.075	-0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	2	S	7500	14500	1600	0.075	-0.125	0.825	0.05	6.5	0.75	1.05
2.5	2	A3	2	D	15000	29000	3200	0.075	-0.125	0.825	0.05	6.5	0.75	1.05
2.5	3	A3	2	S	13250	18250	2550	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A3	2	D	26500	36500	5100	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	4	A3	2	S	19000	22000	3500	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	4	A3	2	D	38000	44000	7000	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	2	S	24750	25750	4450	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	2	D	49500	51500	8900	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	2	S	30500	29500	5400	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	2	D	61000	59000	10800	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	2	S	42000	37000	7300	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	2	D	84000	74000	14600	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	2	S	47750	40750	8250	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	2	D	95500	81500	16500	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	2	S	53500	44500	9200	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	2	D	107000	89000	18400	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	2	S	99500	74500	16800	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	2	D	199000	149000	33600	0.075	-0.125	0.825	0.05	5.75	0.75	1.05

Table E. 2. Hysteretic parameters used for nonlinear analysis

Configuration														
Panel length (ft)	**No. of panels**	**Connector Type**	**No. of connector 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	3375	10875	975	0.075	-0.125	0.825	0.05	8.75	0.75	1.05
2.5	1	A3	3	D	6750	21750	1950	0.075	-0.125	0.825	0.05	8.75	0.75	1.05
2.5	2	A3	3	S	11250	21750	2400	0.075	-0.125	0.825	0.05	6.5	0.75	1.05
2.5	2	A3	3	D	22500	43500	4800	0.075	-0.125	0.825	0.05	6.5	0.75	1.05
2.5	3	A3	3	S	19875	27375	3825	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A3	3	D	39750	54750	7650	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	4	A3	3	S	28500	33000	5250	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	4	A3	3	D	57000	66000	10500	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	3	S	37125	38625	6675	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	5	A3	3	D	74250	77250	13350	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	3	S	45750	44250	8100	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	6	A3	3	D	91500	88500	16200	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	3	S	63000	55500	10950	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	8	A3	3	D	126000	111000	21900	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	3	S	71625	61125	12375	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	9	A3	3	D	143250	122250	24750	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	3	S	80250	66750	13800	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	10	A3	3	D	160500	133500	27600	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	3	S	149250	111750	25200	0.075	-0.125	0.825	0.05	5.75	0.75	1.05
2.5	18	A3	3	D	298500	223500	50400	0.075	-0.125	0.825	0.05	5.75	0.75	1.05

Table E. 3. Hysteretic parameters used for nonlinear analysis

Configuration															
Panel length (ft)	No. of panels	Connector Type	No. of connector 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	5000	8760	1500	0.05	-0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	S	6000	16700	1500	0.1	-0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	S	8000	17670	2000	0.125	-0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	S	10000	21460	2750	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	6	S	12000	25752	3300	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	7	S	14000	30044	3850	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	8	S	16000	34336	3850	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	9	S	18000	38628	4400	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	2	D	10000	17520	3000	0.05	-0.15	0.8	0.05	3.25	0.75	1.02
5	1	A3	3	D	12000	33400	3000	0.1	-0.40	1.05	0.05	4.00	0.60	1.02
5	1	A3	4	D	16000	35340	4000	0.125	-0.20	1.05	0.05	3.50	0.60	1.02
5	1	A3	5	D	20000	42920	5500	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	6	D	24000	51504	6600	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	7	D	28000	60088	7700	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	8	D	32000	68672	7700	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A3	9	D	36000	77256	8800	0.05	-0.15	1.05	0.05	3.75	0.75	1.02

Table E. 4. Hysteretic parameters used for nonlinear analysis

Configuration															
Panel length (ft)	**No. of panels**	**Connector Type**	**No. of connector 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	16666.7	17520.0	3692.3	0.05	-0.15	0.80	0.05	2.41	0.75	1.02
5	2	A3	3	S	20000.0	33400.0	3692.3	0.10	-0.40	1.05	0.05	2.97	0.60	1.02
5	2	A3	4	S	26666.7	35340.0	4923.1	0.13	-0.20	1.05	0.05	2.60	0.60	1.02
5	2	A3	5	S	33333.3	42920.0	6769.2	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	6	S	40000.0	51504.0	8123.1	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	7	S	46666.7	60088.0	9476.9	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	8	S	53333.3	68672.0	9476.9	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	2	D	33333.3	35040.0	7384.6	0.05	-0.15	0.80	0.05	2.41	0.75	1.02
5	2	A3	3	D	40000.0	66800.0	7384.6	0.10	-0.40	1.05	0.05	2.97	0.60	1.02
5	2	A3	4	D	53333.3	70680.0	9846.2	0.13	-0.20	1.05	0.05	2.60	0.60	1.02
5	2	A3	5	D	66666.7	85840.0	13538.5	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	6	D	80000.0	103008.0	16246.2	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	7	D	93333.3	120176.0	18953.8	0.05	-0.15	1.05	0.05	2.79	0.75	1.02
5	2	A3	8	D	106666.7	137344.0	18953.8	0.05	-0.15	1.05	0.05	2.79	0.75	1.02

Table E. 5. Hysteretic parameters used for nonlinear analysis

Configuration															
Panel length (ft)	No. of panels	Connector Type	No. of connector 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	29444.4	22051.0	5884.6	0.05	-0.15	0.80	0.05	2.3	0.8	1.0
5	3	A3	3	S	35333.3	42037.9	5884.6	0.10	-0.40	1.05	0.05	2.8	0.6	1.0
5	3	A3	4	S	47111.1	44479.7	7846.2	0.13	-0.20	1.05	0.05	2.5	0.6	1.0
5	3	A3	5	S	58888.9	54020.0	10788.5	0.05	-0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	6	S	70666.7	64824.0	12946.2	0.05	-0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	7	S	82444.4	75628.0	15103.8	0.05	-0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	8	S	94222.2	86432.0	15103.8	0.05	-0.15	1.05	0.05	2.6	0.8	1.0
5	3	A3	2	D	58888.9	44102.1	11769.2	0.1	-0.2	0.8	0.1	2.3	0.8	1.0
5	3	A3	3	D	70666.7	84075.9	11769.2	0.10	-0.40	1.05	0.05	2.80	0.6	1.0
5	3	A3	4	D	94222.2	88959.3	15692.3	0.13	-0.20	1.05	0.05	2.45	0.6	1.0
5	3	A3	5	D	117777.8	108040.0	21576.9	0.05	-0.15	1.05	0.05	2.63	0.8	1.0
5	3	A3	6	D	141333.33	129648	25892.31	0.05	-0.15	1.05	0.05	2.63	0.75	1.02
5	3	A3	7	D	164888.89	151256	30207.69	0.05	-0.15	1.05	0.05	2.63	0.75	1.02
5	3	A3	8	D	188444.44	172864	30207.69	0.05	-0.15	1.05	0.05	2.63	0.75	1.02

Table E. 6. Hysteretic parameters used for nonlinear analysis

Configuration														
Panel length (ft)	**No. of panels**	**Connector Type**	**No. of connector 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	42222.2	26582.1	8076.9	0.05	-0.15	0.80	0.05	2.14	0.75	1.02
5	4	A3	3	S	50666.7	50675.9	8076.9	0.10	-0.40	1.05	0.05	2.63	0.60	1.02
5	4	A3	4	S	67555.6	53619.3	10769.2	0.13	-0.20	1.05	0.05	2.30	0.60	1.02
5	4	A3	5	S	84444.4	65120.0	14807.7	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	6	S	101333.33	78144	17769.23	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	7	S	118222.22	91168	20730.77	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	8	S	135111.11	104192	20730.77	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	2	D	84444.4	53164.1	16153.8	0.05	-0.15	0.80	0.05	2.14	0.75	1.02
5	4	A3	3	D	101333.3	101351.7	16153.8	0.10	-0.40	1.05	0.05	2.63	0.60	1.02
5	4	A3	4	D	135111.1	107238.6	21538.5	0.13	-0.20	1.05	0.05	2.30	0.60	1.02
5	4	A3	5	D	168888.9	130240.0	29615.4	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	6	D	202666.67	156288	35538.46	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	7	D	236444.44	182336	41461.54	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	4	A3	8	D	270222.22	208384	41461.54	0.05	-0.15	1.05	0.05	2.46	0.75	1.02

Table E. 7. Hysteretic parameters used for nonlinear analysis

Configuration															
Panel length (ft)	No. of panels	Connector Type	No. of connector 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	106111.1	49237.2	19038.5	0.05	-0.15	0.80	0.05	2.14	0.75	1.02
5	9	A3	3	S	127333.3	93865.5	19038.5	0.10	-0.40	1.05	0.05	2.63	0.60	1.02
5	9	A3	4	S	169777.8	99317.6	25384.6	0.13	-0.20	1.05	0.05	2.30	0.60	1.02
5	9	A3	5	S	212222.2	120620.0	34903.8	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	6	S	254666.67	144744	41884.62	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	7	S	297111.11	168868	48865.38	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	8	S	339555.56	192992	48865.38	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	2	D	212222.2	98474.5	38076.9	0.05	-0.15	0.80	0.05	2.14	0.75	1.02
5	9	A3	3	D	254666.7	187731.0	38076.9	0.10	-0.40	1.05	0.05	2.63	0.60	1.02
5	9	A3	4	D	339555.6	198635.2	50769.2	0.13	-0.20	1.05	0.05	2.30	0.60	1.02
5	9	A3	5	D	424444.4	241240.0	69807.7	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	6	D	509333.33	289488	83769.23	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	7	D	594222.22	337736	97730.77	0.05	-0.15	1.05	0.05	2.46	0.75	1.02
5	9	A3	8	D	679111.11	385984	97730.77	0.05	-0.15	1.05	0.05	2.46	0.75	1.02

APPENDIX F: DESIGN EXAMPLE

Description of the Index Buildings

The structure presented here is a multi-family residential structure with a rectangular plan of 40 ft x 60ft. The floor plan is identical for each story and is shown in Figure F.1. Each floor has 4 relatively small apartments and elevator and stair access. Each story is 10ft clear height and the method of construction is platform whereby floor panels bear on and are supported by the vertical CLT panels below. Figure F.2 illustrates assignment of the shear walls utilized as part the lateral force resisting system.

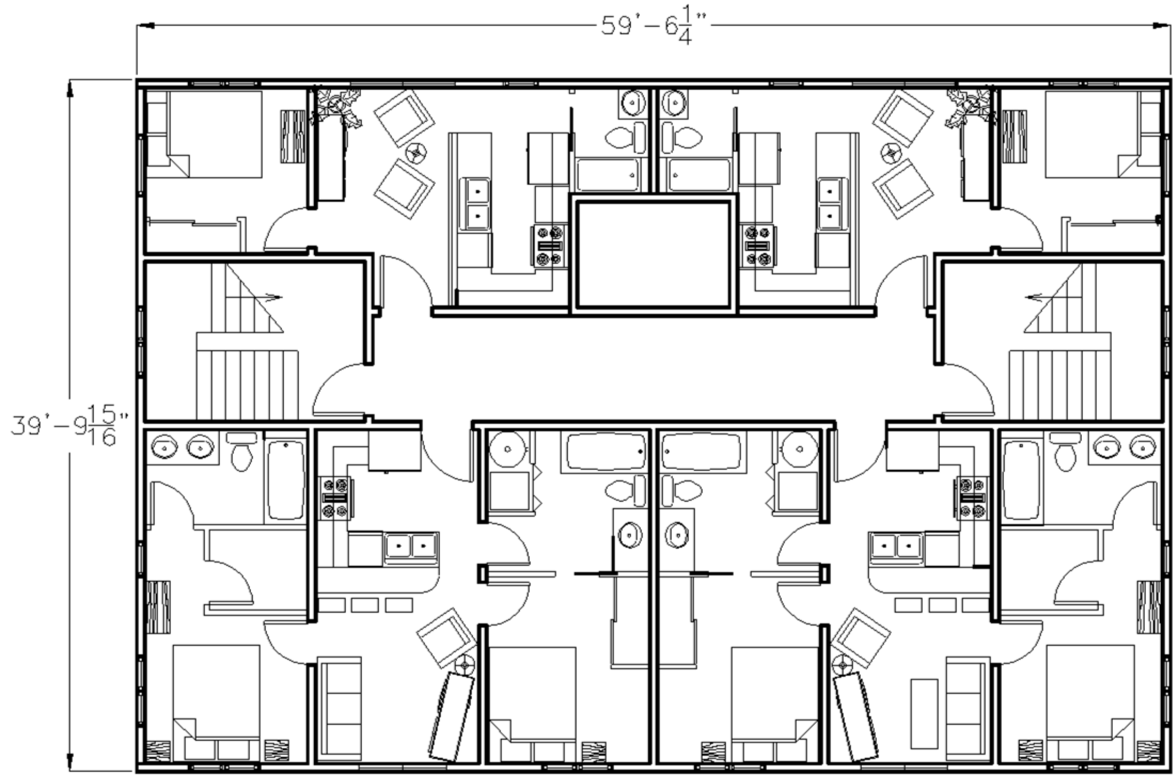


Figure F. 1. Typical floor plan

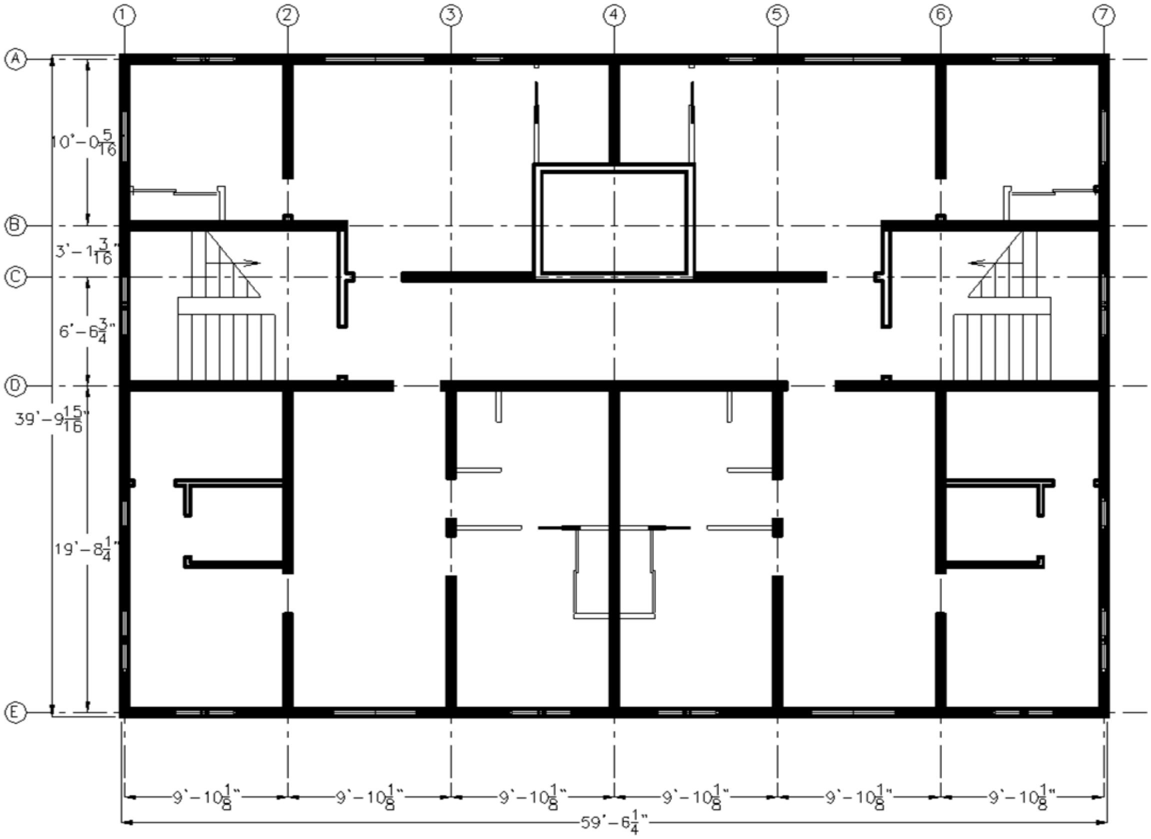


Figure F. 2. Assigned shear wall lines

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. It should be noted that 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 6-story Index Bldg. 4 is provided in Table F.1 and detailed calculations along with the figure are provided in Table F.2 and Figure F.3, respectively.

Table F. 1. Seismic weight summary, Index Bldg. 4

Level	h (ft)	W_{Level} (kip), low gravity	W_{Level} (kip), high gravity
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 F. 2. Seismic weight detailed calculation, Index Bldg. 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} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)	W_{floor} (lbs)	$W_{extWalls}$ (lbs)	$W_{IntWalls}$ (lbs)	ΣW_{Level} (lbs)
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												

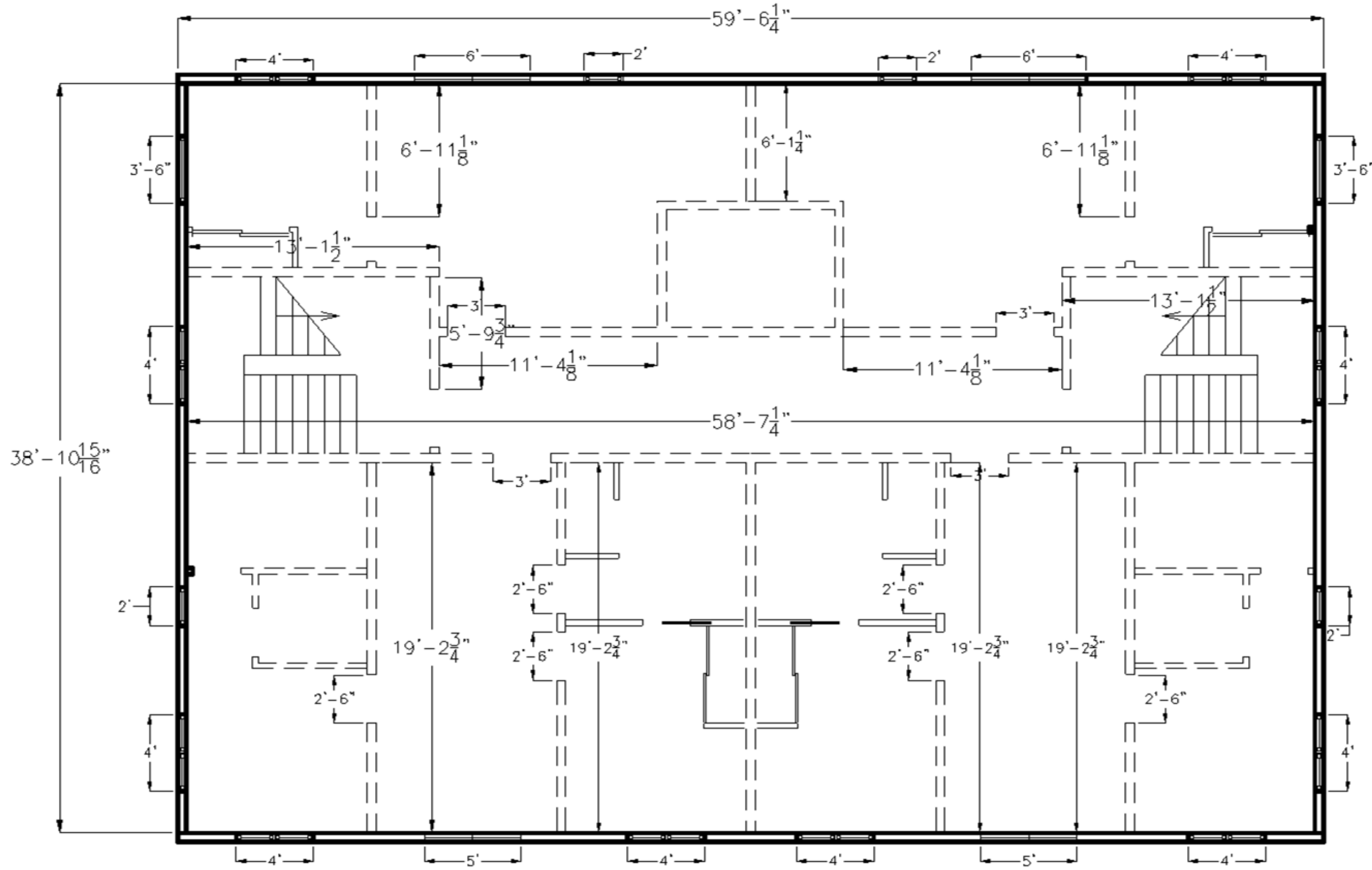


Figure F. 3. Interior and exterior wall dimensions

Archetypes

The design space was divided into various performance groups based on the factors that significantly affect seismic behavior. The list of archetype extracted from shear wall lines 3, B and E of Index Building 4 that falls under different performance groups is given in Table F.3. 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_SDC Dmax or Dmin_Short Period or Long Period

Table F. 3. Extracted Archetypes from Index Bldg. 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

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 6 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, SDC D_{max}, and long period archetype. The tributary area and the extracted archetype are shown in Figures F.4 and F.5, respectively.

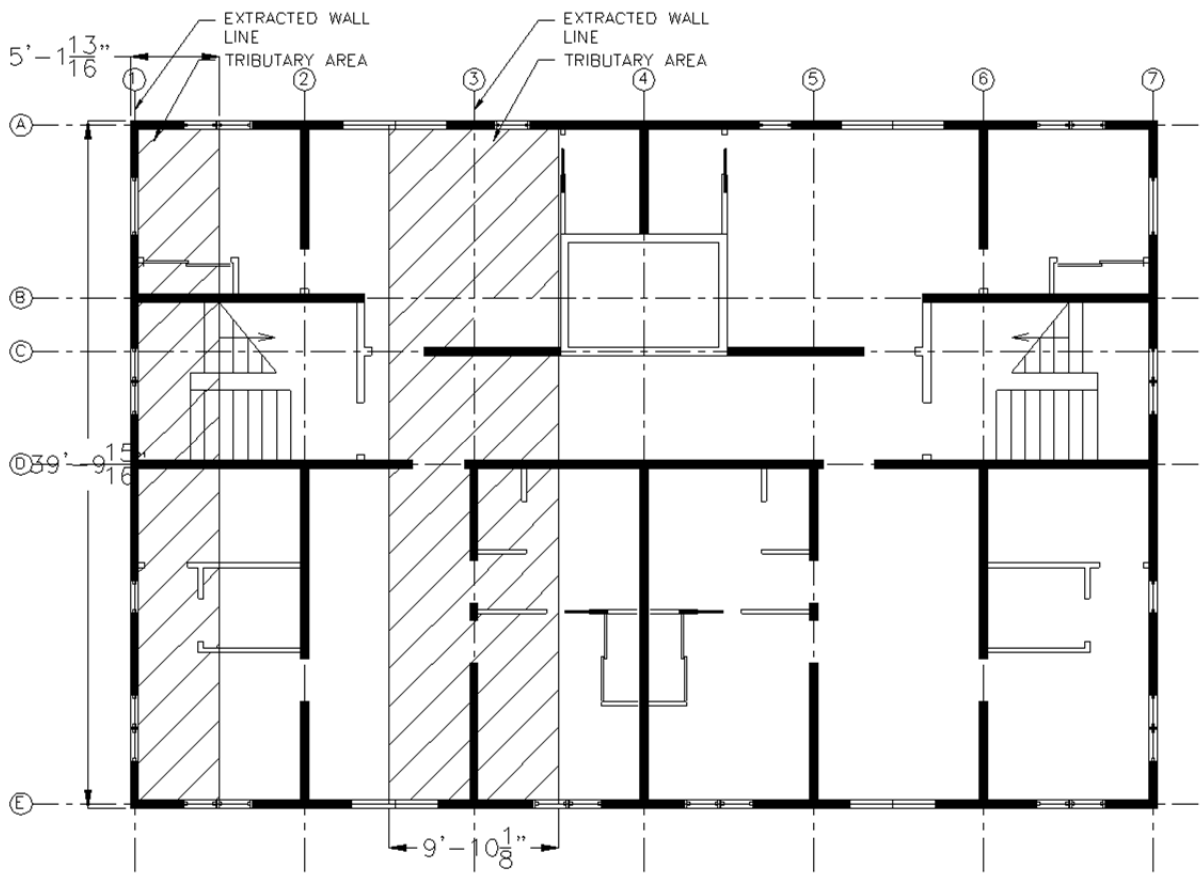


Figure F. 4. Tributary area

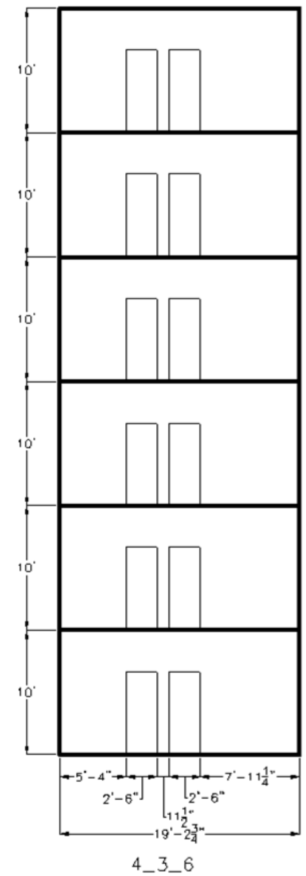


Figure F. 5. Shear wall

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

Seismic load calculations for the wall line are provided in Table F.4 and the tributary area is shown in Figure F.6. The highlighted values in the table were the values used in the seismic load calculations and subsequent design of the archetypes. Two shear walls are assumed to be along the wall line and each one is intended to carry half of the seismic load.

Table F. 4. Tributary load calculation

	Story shear (kip)	Shear wall line	Tributary area of the wall (ft^2)	Fraction of total area	Cumulative Shear Load (kip)
			High gravity		
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

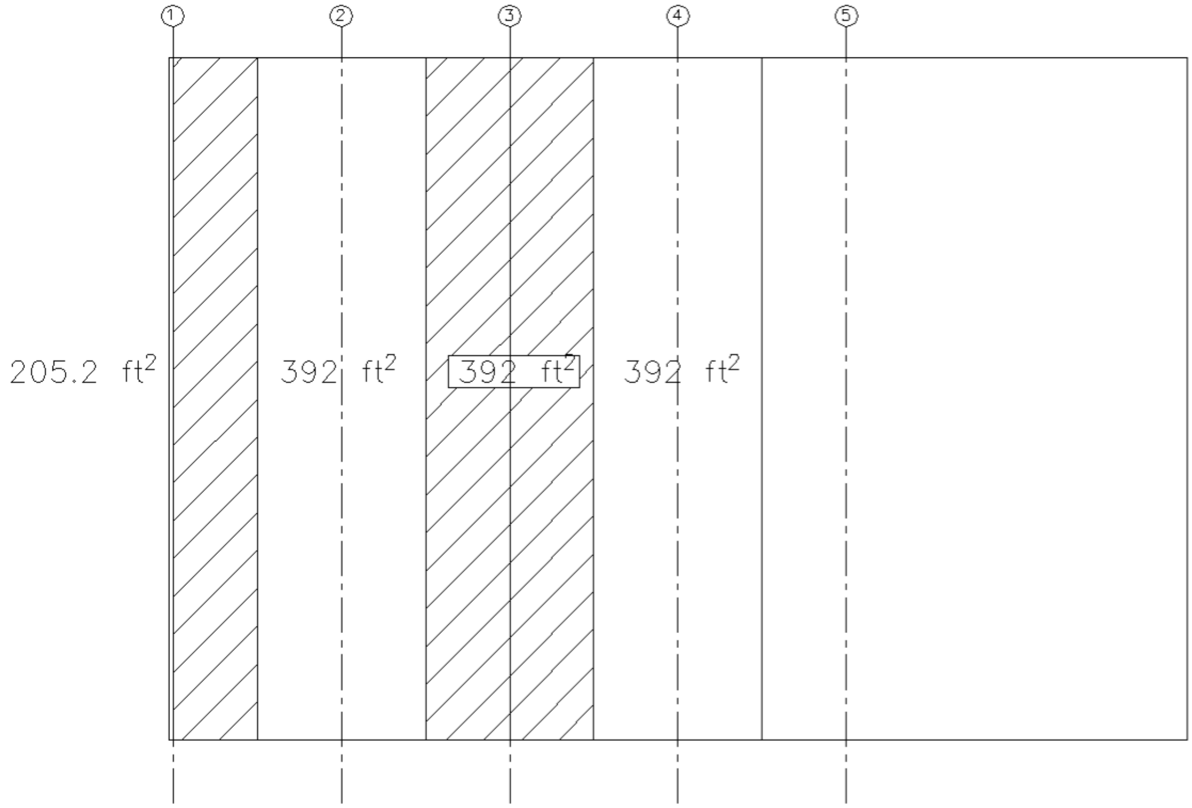


Figure F. 6. Index building 4 seismic tributary area for North-South direction walls

Table F. 5. Archetype tributary seismic load

Story	Seismic Load (kip) *
6	6.4
5	16.0
4	23.7
3	29.3
2	33.0
1	34.8

* These values were obtained by dividing the highlighted values in Table F.4 by 2. This was done since 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 the shear wall line 3.

Archetype Design

With the forces computed, as shown in Table F.6, archetype design was performed based on the design methodology explained in Chapter 6 of this dissertation. Archetypes are to be designed for shear forces as well as the associated overturning moment.

The shear wall shown in Figure F.5 consists of a single 5 ft x 10 ft and (3) 2.5 ft x 10 ft multi-panel configuration. The designed shear wall is shown in Figure F.7 and the following demonstrates design calculations for the 5 ft x 10 ft segment.

- CLT panel size: 5 ft x 10 ft (CLT panel aspect ratio=h/b_s=2:1) per **Section 6.3.1 (b).**
- CLT shear wall at each level: 5 ft x 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 inter-panel connector that is equivalent to Type A connector.
- Number of connectors per panel was chosen to satisfy spacing requirements specified in **Section 6.3.1 (c).**
- Nominal Unit Shear: The unit shear capacity is calculated for single and double sided configuration based on **Table 6.1, Section 6.3.1.**
- Design Unit Shear (LRFD):

  ϕ_D =0.50

Looking at table below based on the design methodology, we have:

Nominal unit shear capacity of CLT special shear walls, plf

Base and top of wall connection	**Vertical joint connection**	**Minimum panel thickness, inch**	**Nominal unit shear capacity, v_s, plf**
Type A (see Table 6.2)	Type E (see Table 6.3)	3.5	v_s = NC*(2605/b) (6.1)

Table F. 6. Archetype Shear Design

No. of Stories	No. of Panels	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)
6	1	5	A3	2	S	521.0	0.41	410.1
5	1	5	A3	3	D	1563.0	1.04	1418.2
4	1	5	A3	5	D	2605.0	1.58	2161.5
3	1	5	A3	5	D	2605.0	1.92	2556.6
2	1	5	A3	5	D	2605.0	1.87	2545.8
1	1	5	A3	6	D	3126.0	2.05	2871.5

No. of Stories	No. of Panels	Panel Length (ft)	Connector type	NC, Number of Connectors/Side/Panel	S/D	Shear Capacity (plf)	Stiffness (kip/in.)	Applied Load (plf)
6	3	2.5	A3	2	S	1042.0	0.88	580.0
5	3	2.5	A3	3	S	1563.0	1.31	1194.2
4	3	2.5	A3	2	D	2084.0	1.88	1715.7
3	3	2.5	A3	3	D	3126.0	2.48	2203.9
2	3	2.5	A3	3	D	3126.0	2.97	2701.8
1	3	2.5	A3	3	D	3126.0	2.92	2723.7

Shear Strength Provided (kip)	Story Shear (kip)/Archetype	Ratio of Provided Shear to Story Shear
10.42	6.40	1.63
19.54	16.05	1.22
28.66	23.68	1.21
36.47	29.31	1.24
36.47	32.99	1.11
39.08	34.79	1.12

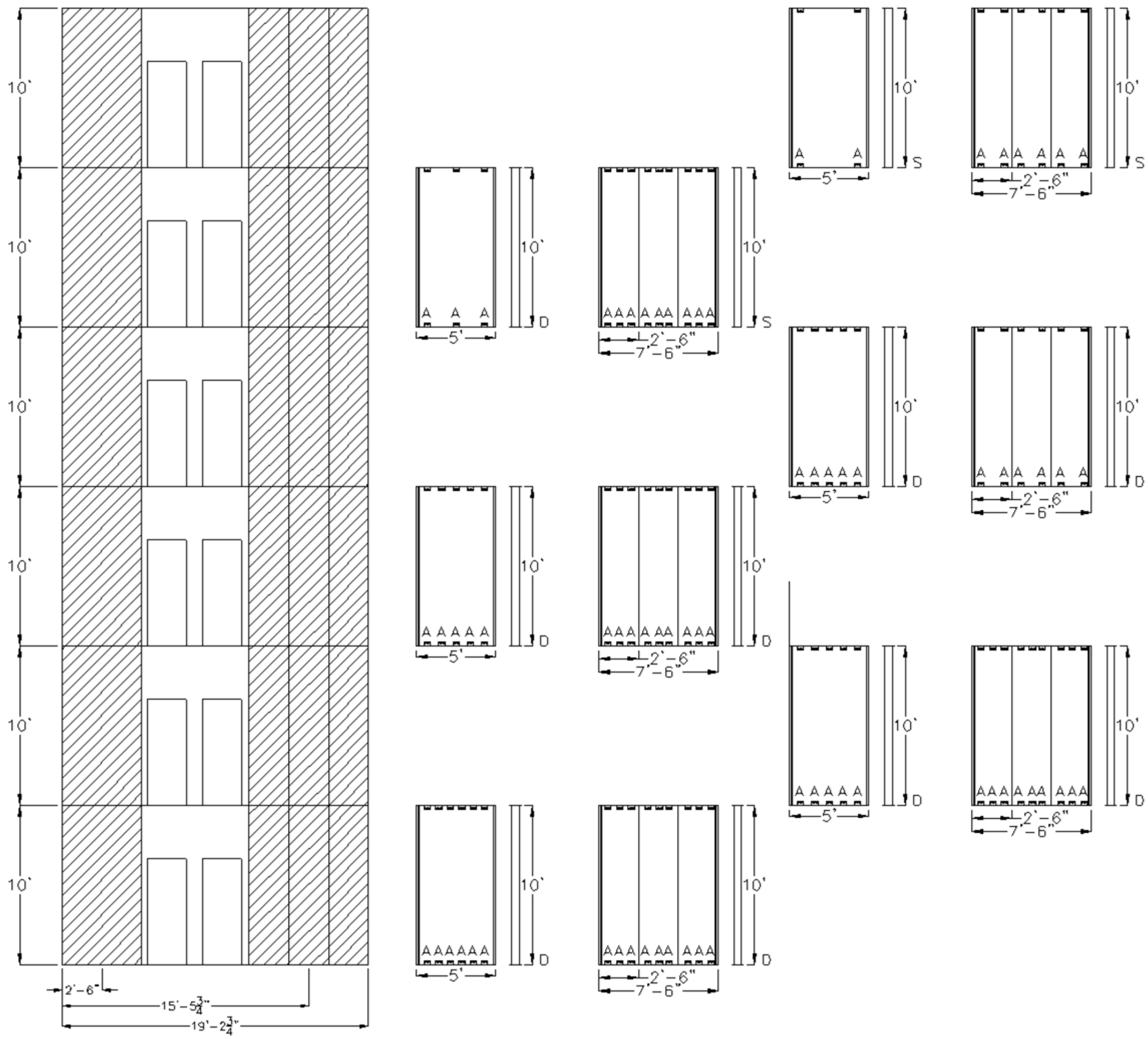
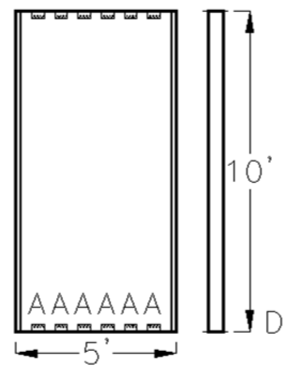


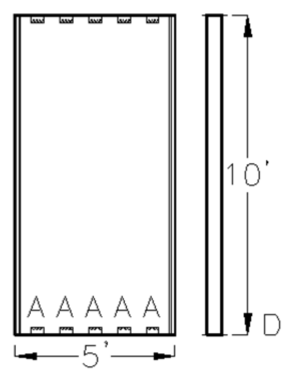
Figure F. 7. Example Archetype Design

Table F. 7. Shear design calculations for CLT panels used in the archetype

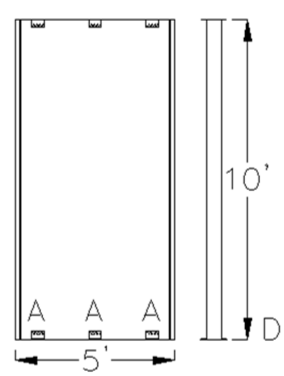
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
5	A3	D	6	=2*6*2605/5=6252	3126



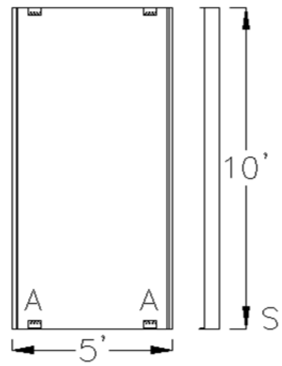
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
5	A3	D	5	=2*5*2605/5=5210	2605



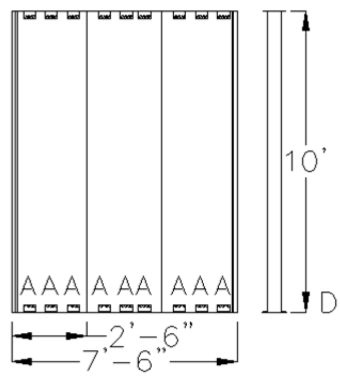
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
5	A3	D	3	=2*3*2605/5=3126	1563



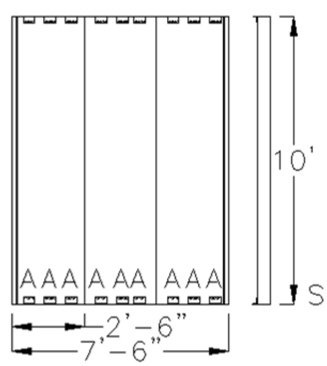
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
5	A3	S	2	=1*2*2605/5=1042	521



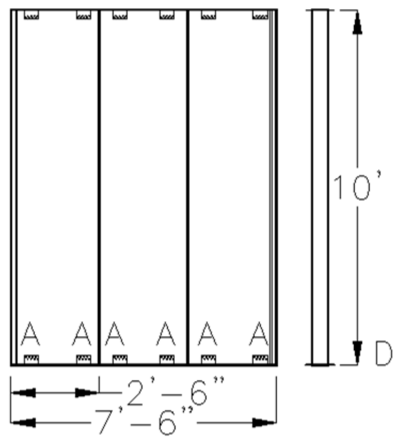
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
2.5	A3	D	2	=2*3*3*2605/7.5=6252	3126



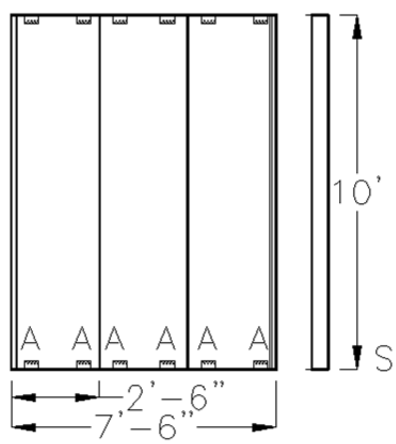
Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
2.5	A3	S	2	=1*3*3*2605/7.5=3126	1563



Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
2.5	A3	D	2	=2*3*2*2605/7.5=4168	2084



Panel Length (ft)	Connector type	S/D	NC, Number of connectors on one side per panel	Nominal Unit Shear Capacity (plf)	Design Unit Shear Capacity (plf)
2.5	A3	S	2	=1*3*2*2605/7.5=2084	1042



- There are no inter-panel connectors for the single panel case. However, for the multi-panel configuration, using simple mechanics and considering the aspect ratio of the individual panels, this results in twice the number of inter-panel connectors as the number of connectors at the base.
- Overturning – Applied unit shear:

  Applied unit shear is required for overturning calculations and is calculated based on the stiffness of each wall within the wall line which is calculated based on Eq. 3 provided in the design methodology.

  The applied unit shear values for the example wall stack are given below.

	Applied unit shear (lb/ft)	
Story	**(1) 5 ft panel**	**(3) 2.5ft panel**
6	410.1	580.0
5	1418.2	1194.2
4	2161.5	1715.7
3	2556.6	2203.9
2	2545.8	2701.8
1	2871.5	2723.7

- 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).

  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

ρ=0

L=0

S=0

D=only wall self-weight; 25.9 psf (Interior wall)

Looking at Figure F.2, 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.

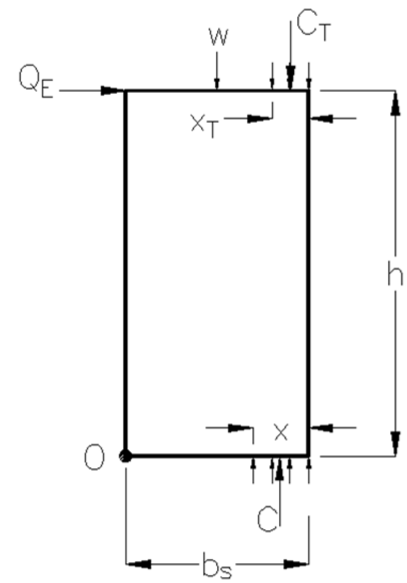
Overturning calculations for the 5ft segment of the wall is shown in the Table below.

No. of Stories	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	0.127	410.09	5.0	3.90	0.15	20.50	22.72	22.72	4.62	3.95	3.95
5	0.317	1418.17	5.0	6.65	0.41	70.91	99.18	99.18	20.68	14.25	18.20
4	0.382	2161.52	5.0	9.41	0.63	108.08	213.94	213.94	45.68	22.46	40.65
3	0.382	2556.57	5.0	9.41	1.09	127.83	348.45	348.45	78.17	28.09	68.74
2	0.446	2545.75	5.0	12.16	1.18	127.29	483.54	483.54	109.61	28.18	96.92
1	0.527	2871.53	5.0	15.63	1.21	143.58	636.34	636.30	144.78	31.87	128.79

No. of Stories	Gravity Load (kip/ft)	1.15*vn Unit Shear (lb/ft)	End panel length (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)
6	0.127	1198.3	5.0	12.0	12.0
5	0.317	3594.9	5.0	36.9	48.9
4	0.382	5991.5	5.0	63.3	112.2
3	0.382	5991.5	5.0	66.6	178.9
2	0.446	5991.5	5.0	67.2	246.1
1	0.527	7189.8	5.0	81.0	327.1

Table F. 8. Overturning Demand-design values for each story

6th Story Shear Wall
$t = 3.9\ in.$
$b = 5ft$
$*w = 25.3\ psf * 5ft * 5ft = 0.632\ k;\ C_T = 0$
$Q_E = 410.09\ plf * 5ft = 2050.45\ lb$
$MO = v * b_s * h = 2050.45\ lb * 10ft = 20.5\ k - ft$
The following equation is solved for x:
$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$
$x = 0.15ft$
$C = 0.638ksi * t * x * \ (12"/ft) = 4.62\ k$
$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * \left(\frac{b_s}{2} - \frac{x}{2}\right)$$
$T = 3.95\ k$
$T_6 = T = 3.95\ k$



*This value was adjusted from the one shown in Appendix A for 3.9 in. wall thickness

5th Story Shear Wall
$t = 6.65\ in.$
$b = 5ft$
$w = 31.8\ psf * 10ft * 5ft = 1.59\ k;\ C_T = 4.62\ k;\ x_T = 0.15ft$
$Q_E = 1418.7plf * 5ft = 7093.5\ lb$
$MO = v * b_s * h = 7093.5\ lb * 10ft = 70.93\ k - ft$
The following equation is solved for x:
$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$
$x = 0.41ft$
$C = 0.638ksi * t * x * \ (12"/ft) = 20.68\ k$
$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * \left(\frac{b_s}{2} - \frac{x}{2}\right)$$
$T = 14.25\ k\ \Rightarrow T_5 = 14.25 + T_6 = 14.25 + 3.95 = 18.20\ k$

4th Story Shear Wall

$t = 9.41\ in.$

$b = 5ft$

$*w = 38.3\ psf * 10ft * 5ft = 1.92\ k;\ C_T = 20.68\ k;\ x_T = 0.41ft$

$Q_E = 2161.52plf * 5ft = 10807.6\ lb$

$MO = v * b_s * h = 10807.6\ lb * 10ft = 108.08\ k - ft$

The following equation is solved for x:

$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$

$x = 0.63ft$

$C = 0.638ksi * t * x * \ (12"/ft) = 45.68\ k$

$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * (\frac{b_s}{2} - \frac{x}{2})$$

$T = 22.46\ k\ \Rightarrow T_4 = 22.46 + T_5 = 22.46 + 14.25 = 40.65\ k$

*This value was adjusted from the one shown in Appendix A for 9.41 in. wall thickness

3rd Story Shear Wall

$t = 9.41\ in.$

$b = 5ft$

$*w = 38.3\ psf * 10ft * 5ft = 1.92\ k;\ C_T = 45.68\ k;\ x_T = 0.63ft$

$Q_E = 2556.57plf * 5ft = 12782.8\ lb$

$MO = v * b_s * h = 12782.8\ lb * 10ft = 127.83\ k - ft$

The following equation is solved for x:

$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$

$x = 1.09ft$

$C = 0.638ksi * t * x * \ (12"/ft) = 78.17\ k$

$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * (\frac{b_s}{2} - \frac{x}{2})$$

$T = 28.09k \Rightarrow T_3 = 28.09 + T_4 = 28.09 + 40.65 = 68.74\ k$

*This value was adjusted from the one shown in Appendix A for 12.16 in. wall thickness

2nd Story Shear Wall

$t = 12.16\ in.$

$b = 5ft$

$w = 44.6\ psf * 10ft * 5ft = 2.23\ k;\ C_T = 78.17\ k;\ x_T = 1.09ft$

$Q_E = 2545.75 * 5 = 12728.75\ lb$

$MO = v * b_s * h = 12728.75\ \ lb * 10ft = 127.29\ k - ft$

The following equation is solved for x:

$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$

$x = 1.18ft$

$C = 0.638ksi * t * x *\ (12"/ft) = 109.61\ k$

$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * (\frac{b_s}{2} - \frac{x}{2})$$

$T = 28.18\ k\ \Rightarrow T_2 = 28.18 + T_3 = 28.18 + 68.74 = 96.92\ k$

*This value was adjusted from the one shown in Appendix A for 15.63 in. wall thickness

1st Story Shear Wall

$t = 15.63\ in.$

$b = 5ft$

$w = 52.8\ psf * 10ft * 5ft = 2.64\ k;\ C_T = 109.61\ k;\ x_T = 1.18ft$

$Q_E = 2871.53 * 5 = 14357.65\ lb$

$MO = v * b_s * h = 14357.65\ \ lb * 10ft = 143.58\ k - ft$

The following equation is solved for x:

$$v * b_s * h + 1.4 * w * b_s * \frac{b_s}{2} + C_T * \left(b_s - \frac{x_T}{2}\right) = 0.638ksi * t * x * \left(b_s - \frac{x}{2}\right)$$

$x = 1.21ft$

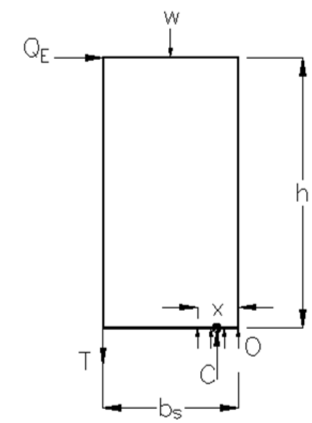
$C = 0.638ksi * t * x *\ (12"/ft) = 144.78\ k$

$$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * (\frac{b_s}{2} - \frac{x}{2})$$

$T = 31.87\ k\ \Rightarrow T_1 = 31.87 + T_2 = 31.87 + 96.92 = 128.79\ k$

Table F. 9. Overturning Demand-nominal values for each story

1st Story Shear Wall
$b = 5ft$
$w = 52.8\, psf * 10ft * 5ft = 2.64$
$1.15V_N = 7198.8plf * 5ft = 35994\ lb$
$MO = v * b_s * h = 35994\ \ lb * 10ft = 359.94\ k - ft$
$x = 1.21ft$
$T * \left(b_s - \frac{x}{2}\right) = v * b_s * h - 0.7 * w * b_s * (\frac{b_s}{2} - \frac{x}{2})$
$T = 81.0\ k\ \ \Rightarrow T_1 = 81.0 + T_2 = 81.0 + 246.1 = 327.1\ k$
T_2 is the accumulated tensile load on the second story corresponding to the nominal capacities of the stories above times 1.15.



Similar calculations were performed for the remaining stories and tensile forces are added at each level.

The design overturning forces are summarized in Table F.10.

Sec. 6.3.2.2 of the methodology requires:

- Rods at each level are designed for cumulative overturning tensile forces and bearing for rod forces shall be provided at each story
- The nominal strength of the tie-down device shall not be less than required to resist the net uplift forces associated with development of the maximum expected shear wall unit shear capacity where expected shear wall unit shear capacity is taken as 1.15 times the nominal unit shear capacity

Tie-down design calculations are provided in Table F.11 and bearing forces are calculated in Table F.12. 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 F. 10. Design Overturning Forces Summary

			Design		Nominal	
No. of Stories	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	4.62	0.15	3.95	3.95	12.0	12.0
5	20.68	0.41	14.25	18.20	36.9	48.9
4	45.68	0.63	22.46	40.65	63.3	112.2
3	78.17	1.09	28.09	68.74	66.6	178.9
2	109.61	1.18	28.18	96.92	67.2	246.1
1	144.78	1.21	31.87	128.79	81.0	327.1

Table F. 11. Tie-down rod design

							Design		Ultimate Capacity Design		Drift Design	
No. of Stories	No. of rods	Dia. (in.)	Nominal Area (in^2)	Net Area (in^2)	Fy (psi)	Fu (psi)	Design Strength, ØRn (kip)	T (kip) Demand	Rod Ultimate strength, Rn (kip)	T corresponding to Wall ultimate capacity	T (kip)	Rod elongation (in.)
6	4	3/8	0.110	0.078	36000	58000	14.41	3.9	19.2	11.95	3.9	0.05
5	4	1/2	0.196	0.142	92000	120000	53.01	18.2	70.7	48.89	18.2	0.13
4	4	3/4	0.442	0.334	92000	120000	119.28	40.7	159.0	112.24	40.7	0.13
3	4	1 1/8	0.994	0.763	92000	120000	268.39	68.7	357.8	178.86	68.7	0.09
2	4	1 1/4	1.227	0.969	92000	120000	331.34	96.9	441.8	246.10	96.9	0.10
1	4	1 3/8	1.485	1.155	92000	120000	400.92	128.8	534.6	327.09	128.8	0.12

Table F. 12. Bearing forces at each story

		Drift Design	
No. of Stories	No. of rods	T corresponding to Wall ultimate capacity LRFD cumulative tension (kip)	Bearing/plate (kip)
6	4	11.95	12.0
5	4	48.89	36.9
4	4	112.24	63.3
3	4	178.86	66.6
2	4	246.10	67.2
1	4	327.09	81.0

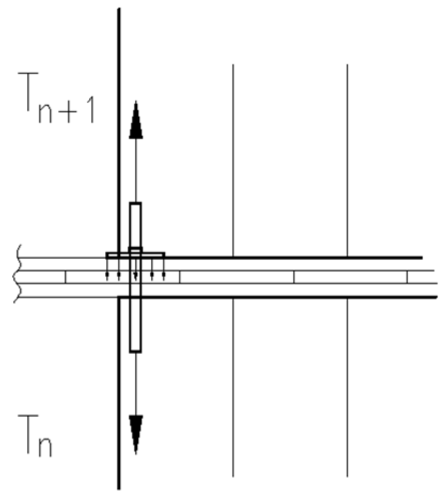


Figure F. 8. Bearing forces at each story

Table F. 13. Deflection calculation- rocking component

Modulus of Elasticity, Eo (psi)	E_perp, E/30 (psi)	Modulus of Elasticity, E90 (psi)	E_perp, E90/30 (psi)	Floor Thickness (in.)	Elastic comp. toe fc_perp deformatin (in.)	Elastic tension side fc_perp deformatin (in.)	Rod elongation (Table F.11) (in.)	$\frac{h}{b}$	Δa, Total vertical elongation of the wall anchorage system*	$\Delta_a \frac{h}{b}$
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.05	2.0	0.24	0.48
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.13	2.0	0.33	0.66
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.13	2.0	0.33	0.66
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.09	2.0	0.31	0.62
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.10	2.0	0.32	0.65
1400000	46666.67	1200000	40000	6.65	0.09	0.09	0.12	2.0	0.34	0.68

*Rod elongation scaled to reflect anchorage deformation at the panel edge

Table F. 14. Deflection calculation- sliding component

# nails/ ft	Load per nail, vn (lb)	Nail diameter, D (in.)	Sliding deflection $v_n/(135{,}000\ D^{1.5})$ (in.)	CLT panel sliding due to horiz. Joint slip (in.)	CLT panel aspect ratio	CLT panel rotation due to vertical joint slip (in.)	Sliding + Rotation due to joint slip (in.)
3.2	128.15	0.135	0.02	0.06	2	0.00	0.06
9.6	147.73	0.135	0.02	0.07	2	0.00	0.07
16	135.09	0.135	0.02	0.06	2	0.00	0.06
16	159.79	0.135	0.02	0.07	2	0.00	0.07
16	159.11	0.135	0.02	0.07	2	0.00	0.07
19.2	149.56	0.135	0.022	0.07	2	0.00	0.07

Table F. 15. Deflection calculation- bending component

Number of layers	I, $b^3a_m/12$ (in^4)	$(EI)_{eff}$ (lb-in^2)	Deflection due to bending $Ph^3/3EI$ (in.)
3	70200	66456000000	0.02
5	119700	1.02463E+11	0.04
7	169380	1.38408E+11	0.04
7	169380	1.38408E+11	0.05
7	218880	1.78856E+11	0.04
7	281340	2.29895E+11	0.04

Table F. 16. Deflection calculation summary

				C_d=3	
Deflection due to bending $Ph^3/3EI$ (in.)	Sliding + Rotation due to joint slip (in.)	$\Delta_a \frac{h}{b}$	Total deflection (in.),	$C_d*\delta$ (in.)	Drift (%)*
0.02	0.06	0.48	0.55	1.65	1.38
0.04	0.07	0.66	0.76	2.28	1.90
0.04	0.06	0.66	0.76	2.29	1.91
0.05	0.07	0.62	0.74	2.23	1.86
0.04	0.07	0.65	0.76	2.28	1.90
0.04	0.07	0.68	0.78	2.34	1.95

The inter-story drift ratio is checked against the drift limit of ASCE 7 which is 2%.

- Deflection calculation for 1st story:

$$\Delta = \frac{576 v b_s h^3}{EI_{eff}} + 3\Delta_{nail\ slip,h} + 2\,\Delta_{nail\ slip,v}\frac{h}{b_s} + \Delta_a\frac{h}{b}$$

Bending:

$\delta_b = 576 v b_s h^3/(EI\_eff)$

$$EI_{eff} = \left[1-\left(1-\frac{E_{90,T}}{E_{0,L}}\right)\frac{a_{m-2}-a_{m-4}+\cdots\pm a_1}{a_m}\right]E_{0,L} * \frac{bpanel^3 * a_m}{12}$$

$$EI_{eff} = \left[1-\left(1-\frac{40000\ psi}{1400000\ psi}\right)\frac{11.16\ in-6.7\ in+2.23\ in}{15.63}\right]1400000\ psi * \frac{\left(5ft*\frac{12in}{ft}\right)^3 * 15.63\ in}{12} = 2.299E11\ lb-in^2$$

$$\delta_b = 576 * 2871.53\ plf * 5ft * \frac{(10ft)^3}{2.299E11\ \ lb-in^2} = 0.04\ in.$$

Shear (Sliding):

$$\Delta_{nail\ slip,h} = \frac{V_n}{6700} = \frac{156.27}{6700} = 0.0233\ in.$$

$$V_n = \frac{2871.53\ plf*5ft}{2*6\ conn*\frac{8\ nails}{conn}} = 149.56\ lb/nail$$

$$\delta_s = 3\Delta_{nail\ slip,h} = 3*0.0233\ in = 0.07\ in.$$

Panel rotation due to vertical joint slip:

$$\Delta_{nail\ slip,v} = 0$$

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 psi
E_{perp}= E_0/30= 1,400,000 psi/30=46,666.67 psi

$$\Delta_1 = \frac{PL}{AE} = \frac{C * t_{floor}}{A_c * E_{perp}} = \frac{144.78\ kip * 1000 lb/kip * 6.65\ in}{\left(1.21 ft * 12\frac{in}{ft} * 15.63\ in\right) * 46666.67 psi} = 0.09\ in.$$

$$\Delta_2 = \frac{PL}{AE} = \frac{C * t_{floor}}{A_c * E_{perp}} = \frac{144.78\ kip * 1000 lb/kip * 6.65\ in}{\left(1.21 ft * 12\frac{in}{ft} * 15.63\ in\right) * 46666.67 psi} = 0.09\ in.$$

$$\Delta_3 = \frac{PL}{A_{net}E} = \frac{T * L}{A_{rod} * E} = \frac{128.79\ k * 120 in}{4 * 1.155\ in^2 * 29000 ksi} = 0.12\ in.$$

$$\mathit{\Delta}_a = (\Delta_1 + \Delta_2 + \Delta_3)\frac{\sum b_s}{\bar{b}} = (0.09 + 0.09 + 0.12) * \frac{5}{(5 - \frac{1.21}{2})}] = 0.34\ in.$$

$$\delta_r = \mathit{\Delta}_a \frac{h}{b} = 0.34 * 2 = 0.68$$

$$\boldsymbol{\delta_{sw} = \delta_b + \delta_s + \delta_R = 0.04 + 0.07 + 0.68 = 0.78\ in.}$$

$$\boldsymbol{C_d \delta_{sw} = 3 * 0.78 \frac{in}{120 in} = 2.34}\%$$

It is important to note that this archetype example similar to any design problem was performed in an iterative manner where variables such as number of connectors, wall thicknesses, tie-down numbers and sizes were refined.

Archetype design properties are summarized in Table F. 17. Based on the final design the following CUREE parameters, shown in Table F.18, were used for static pushover and dynamic analysis.

Table F. 17. Archetype Example Design Properties

No.	Archetype ID	Key Archetype Design Parameters							
		Panel aspect ratio	Seismic Design Criteria						S_{MT}(g)
			Gravity	SDC	T(sec)	T_1(sec)	V_b(kip)	W(kip)	
52	4_3_ 6_1_MR_HG_ DX_LP	Mix	High	D_{max}	0.604	0.72	34.79	105	1.49

Table F. 18. CUREE parameters used for example archetype

Configuration															
Panel length (ft)	No. of panels	Connector Type	No. of connector per panel	S/D	K_0	F_0	F_1	r_1	r_2	r_3	r_4	Δ_u	α	β
5	1	A	6	D	24000	51504	6600	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A	5	D	20000	42920	5500	0.05	-0.15	1.05	0.05	3.75	0.75	1.02
5	1	A	3	D	12000	33400	3000	0.1	-0.4	1.05	0.05	4	0.6	1.02
5	1	A	2	S	5000	8760	1500	0.05	-0.15	0.8	0.05	3.25	0.75	1.02
2.5	3	A	2	D	26500	36500	5100	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A	2	S	13250	18250	2550	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A	3	D	39750	54750	7650	0.075	-0.125	0.825	0.05	6.125	0.75	1.05
2.5	3	A	3	S	19875	27375	3825	0.075	-0.125	0.825	0.05	6.125	0.75	1.05

Monotonic static pushover analysis was performed for the archetype using the inverted triangular load distribution. Loads were applied proportionate to the fundamental mode of the structure. The pushover curve for the example archetype is shown in Figure F.9. From inspection of the figure, 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.

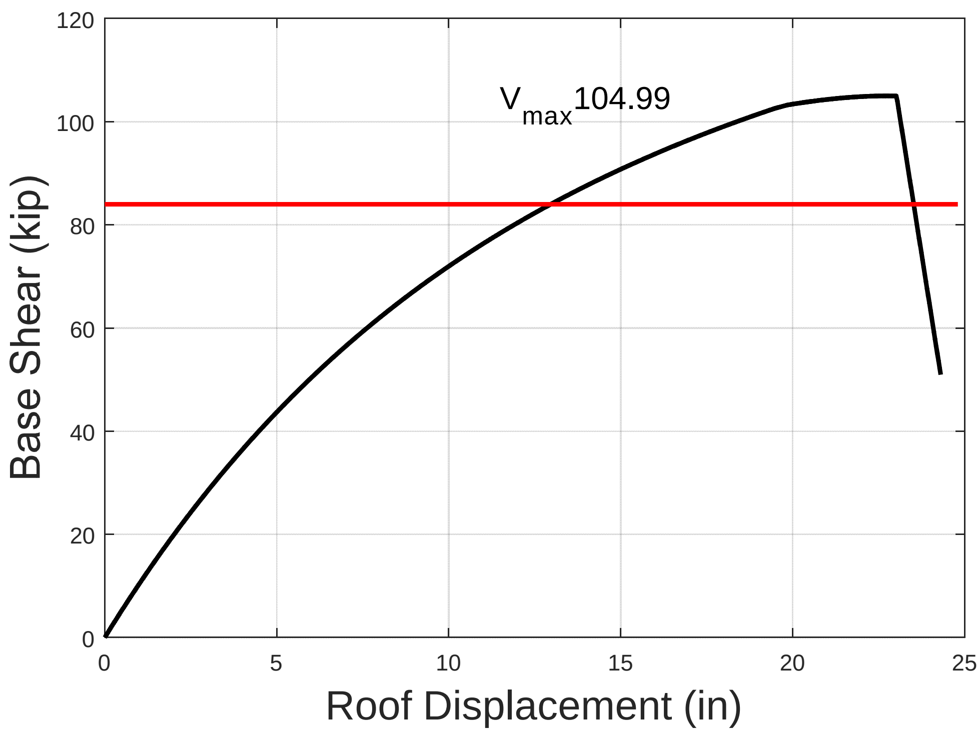
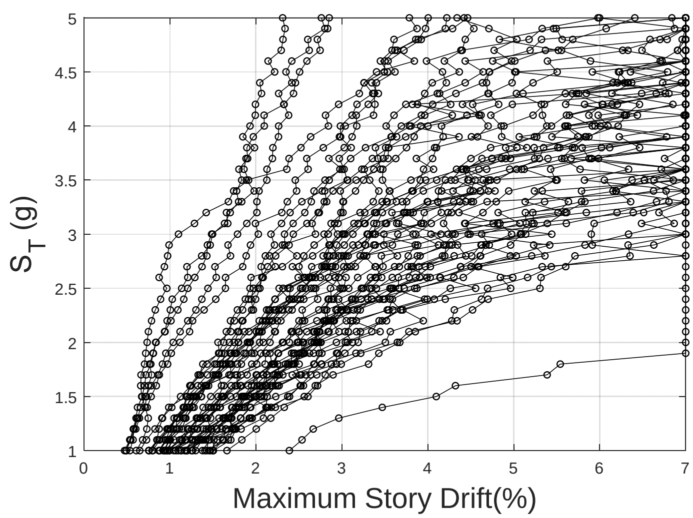


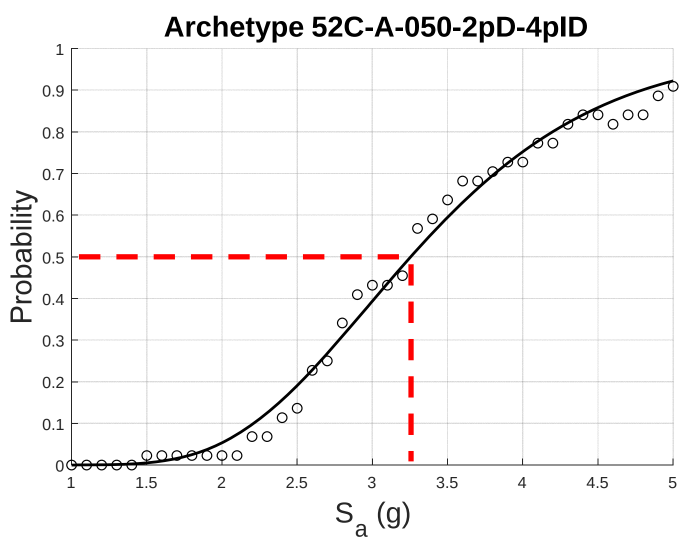
Figure F. 9.Archetype 53, R=3, static pushover

As explained in Section 7.4, Incremental Dynamic Analysis was performed in order to determine the median collapse intensity, $\hat{S}_{CT}$. An IDA plot and collapse fragility for the example archetype are shown in Figure F.10a and F.10b, respectively. The latter was constructed based on 4% inter-story drift limit discussed in Section 8.1. Looking at the figure, $\hat{S}_{CT}$ = 3.25 g where the collapse margin ratio, CMR, taken as the ratio of $\hat{S}_{CT}$ and MCE spectral acceleration, S_{MT}, is calculated as

3.25g/1.49g=2.18. A summary of the static and dynamic analyses are provided in Table F.19. 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 ACMR requires total collapse uncertainty which was calculated based on the explanation provided in Section 8.2. 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 ACMR$_{20\%}$ of 1.56.



(a)



(b)

Figure F. 10. Archetype 52 R=3, 2% damping, (a) IDA plot (b) collapse fragility based on 4% interstory drift limit

Table F. 19. Summary of Collapse Results for Archetype Example

No.	Archetype ID	Design Configuration			Collapse Margin Parameters						Acceptance Check	
		Panel aspect ratio	Gravity	Seismic SDC	Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	ACMR	Acceptable ACMR	Pass/Fail
52	4_3_6_1_MR_HG_DX_LP	Mix	High	D_{max}	3.02	3.42	3.25	2.18	1.18	2.57	1.56	PASS

APPENDIX G: ARCHETYPE RESULTS

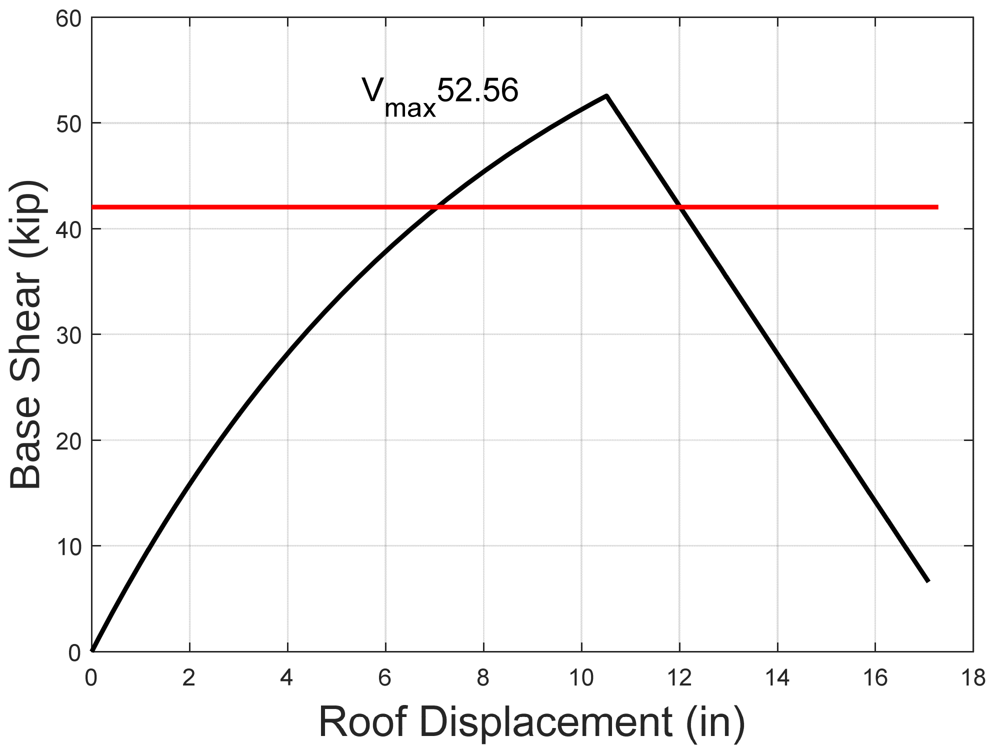


Figure G. 1. Archetype 01, R=3, static pushover

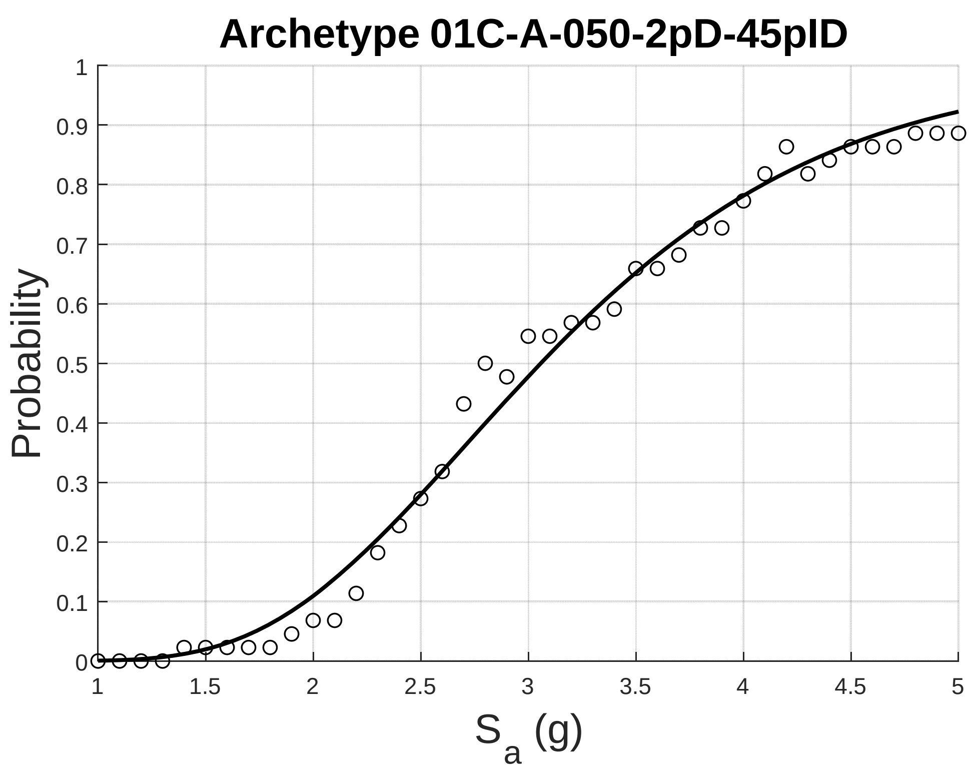


Figure G. 2. Archetype 01 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

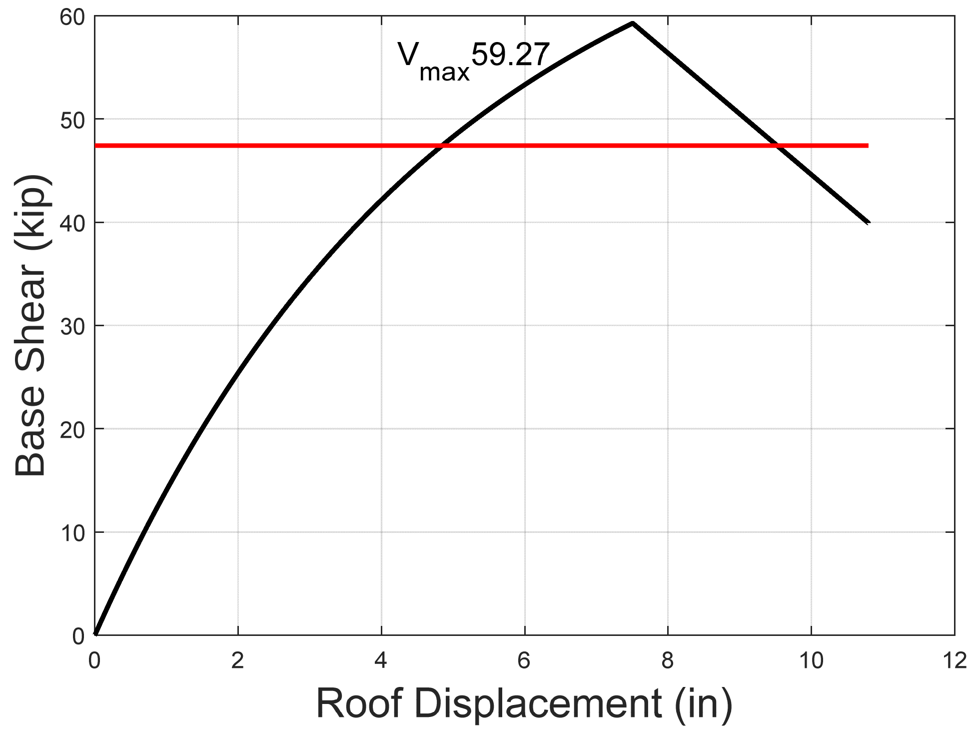


Figure G. 3. Archetype 02, R=3, static pushover

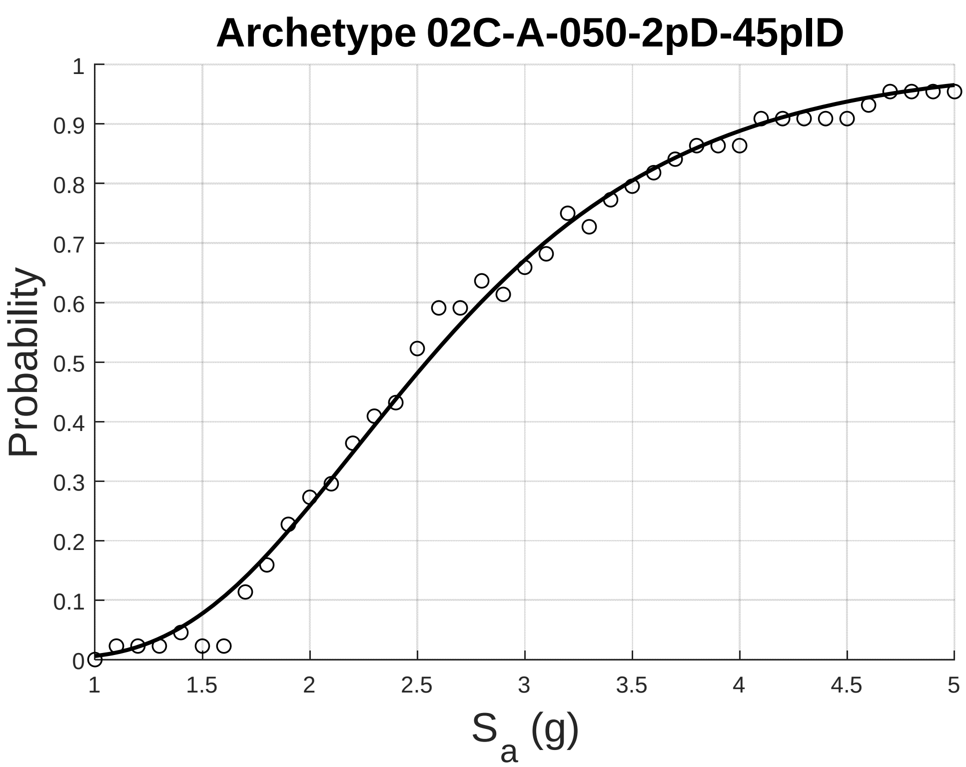


Figure G. 4. Archetype 02 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

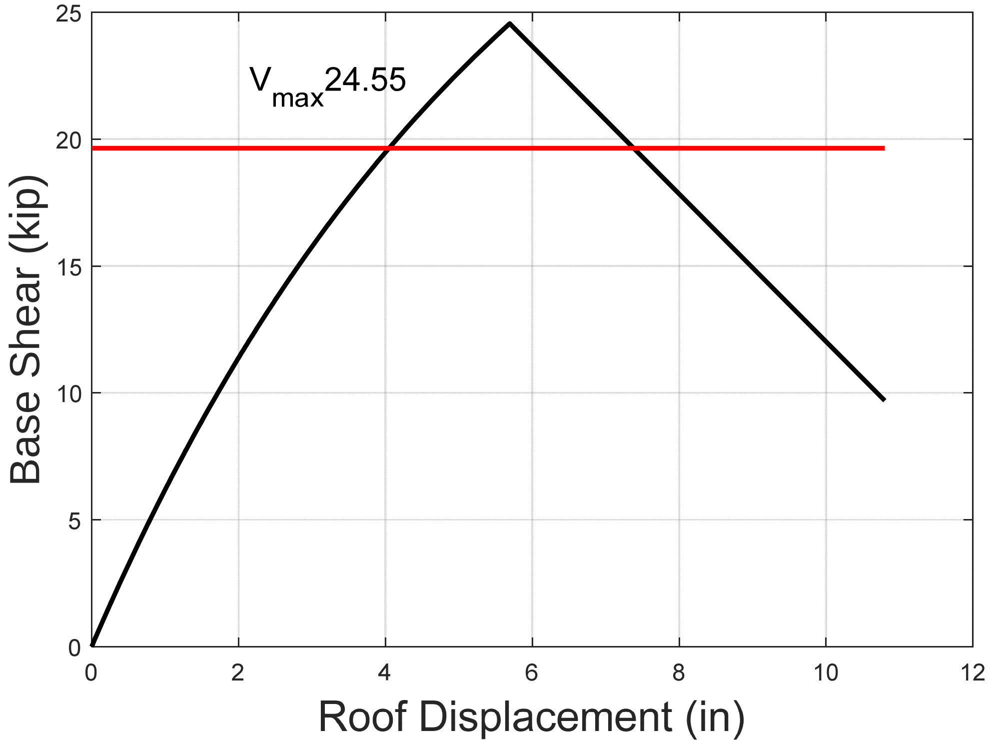


Figure G. 5. Archetype 03, R=3, static pushover

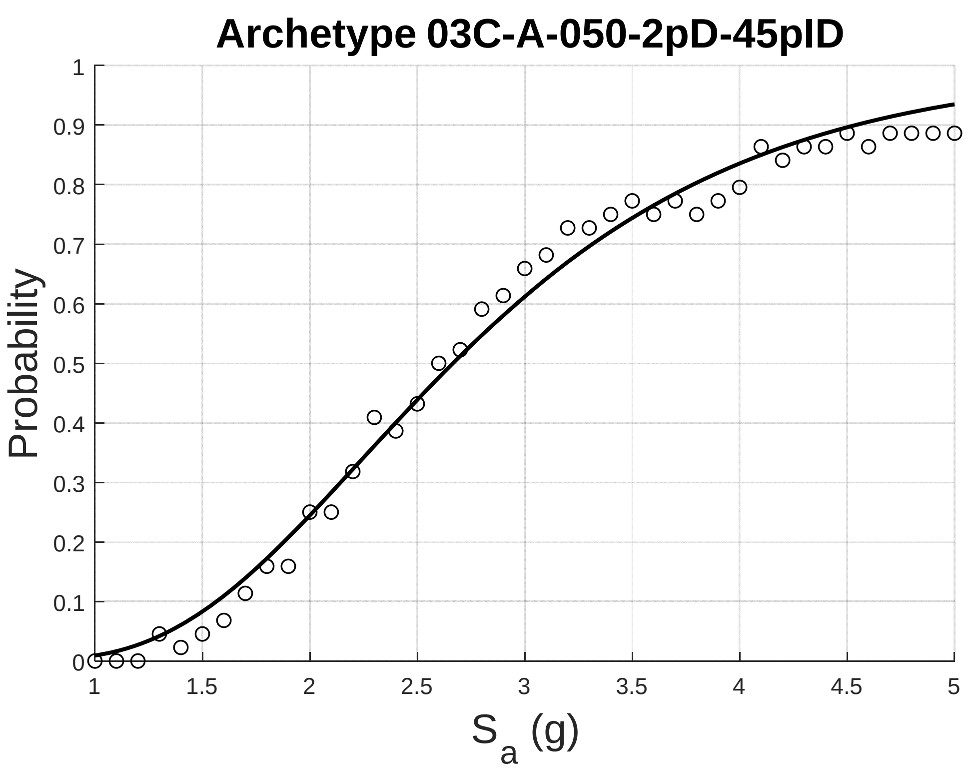


Figure G. 6. Archetype 03 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

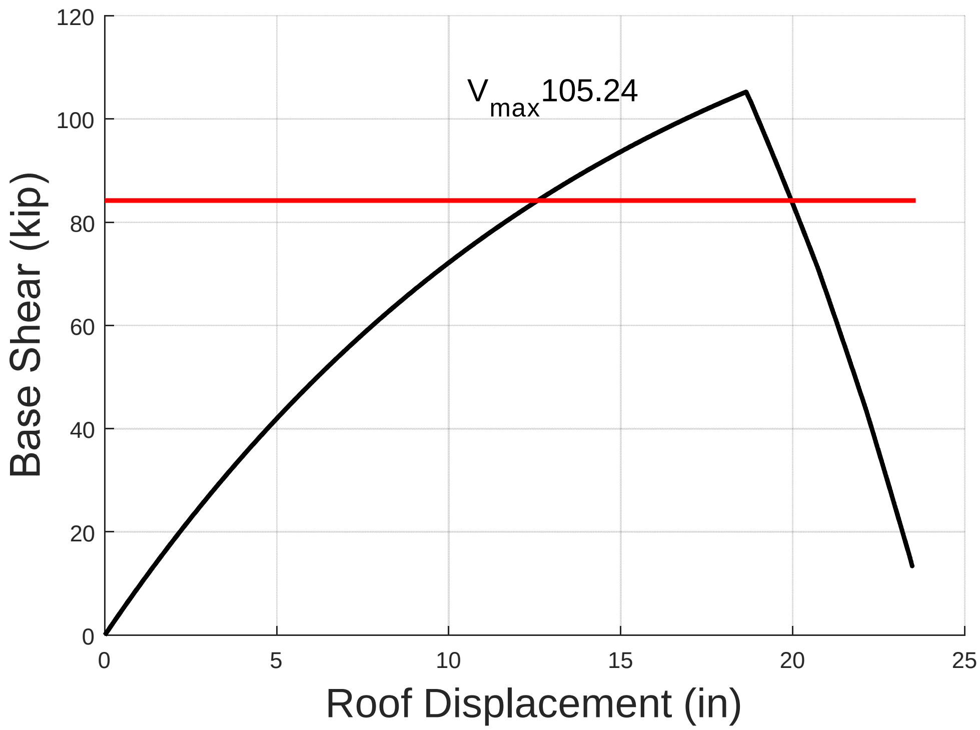


Figure G. 7. Archetype 04, R=3, static pushover

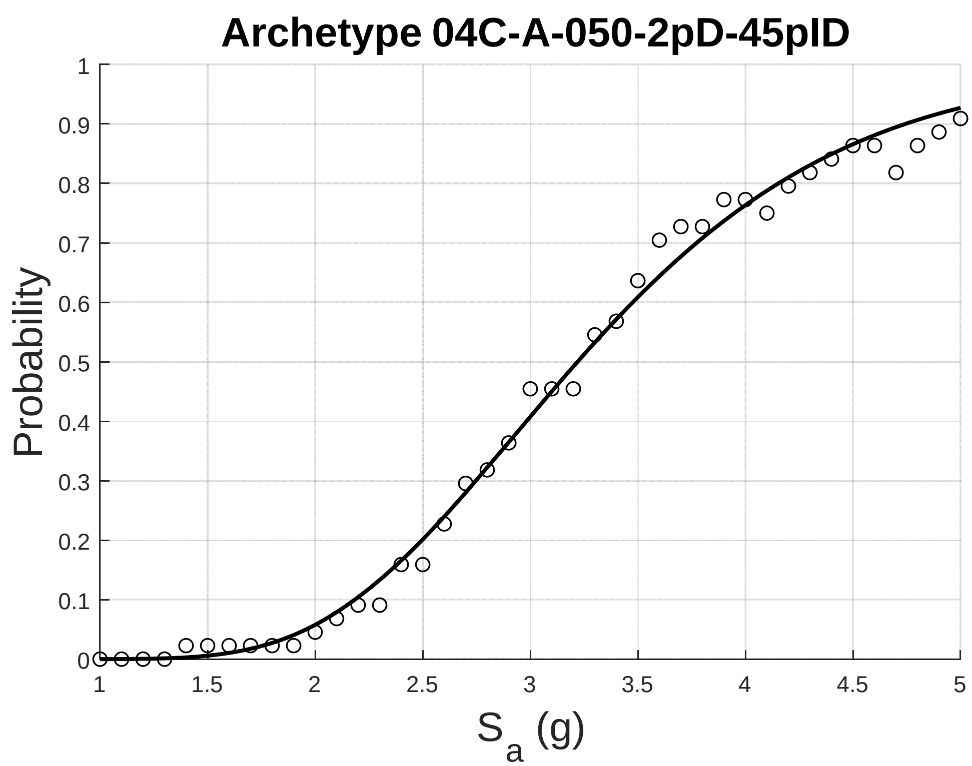


Figure G. 8. Archetype 04 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

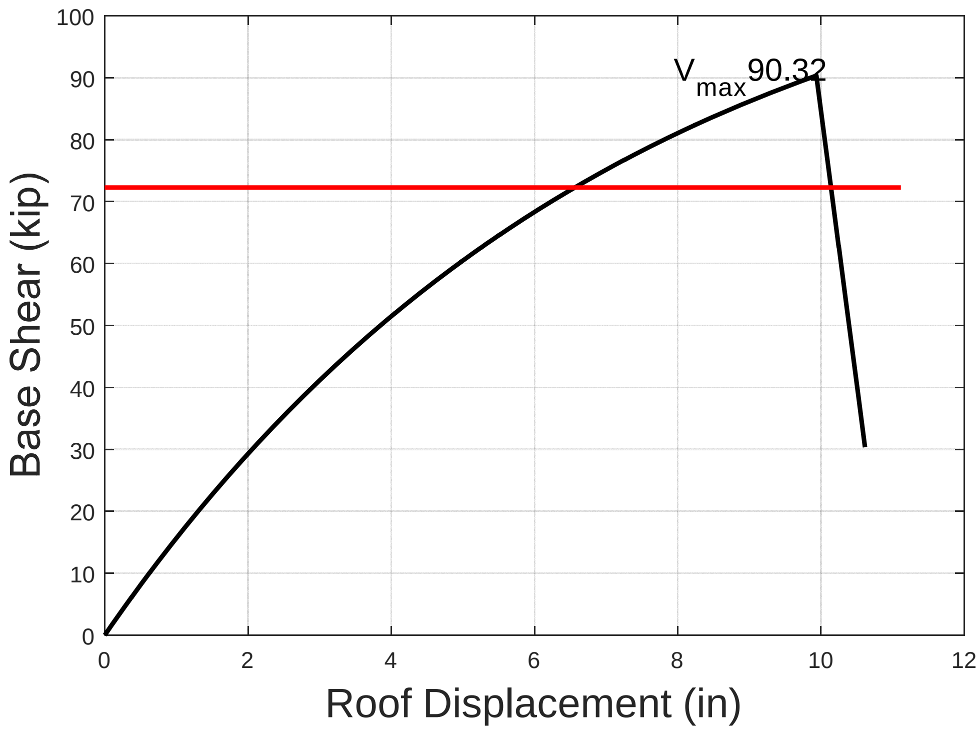


Figure G. 9. Archetype 05, R=3, static pushover

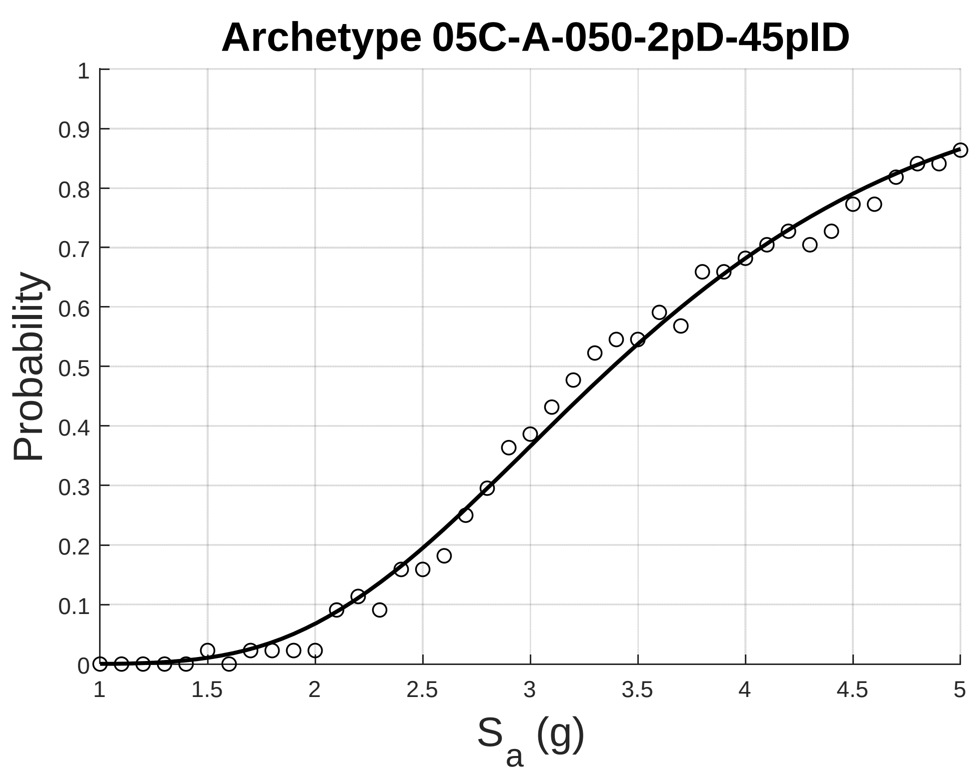


Figure G. 10. Archetype 05 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

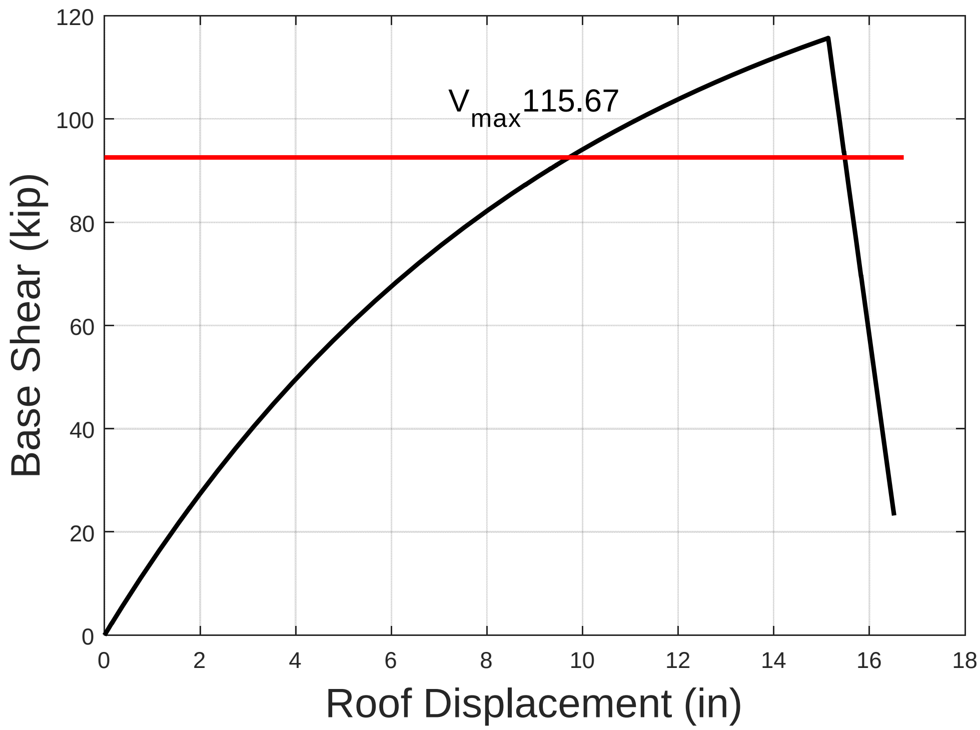


Figure G. 11. Archetype 06, R=3, static pushover

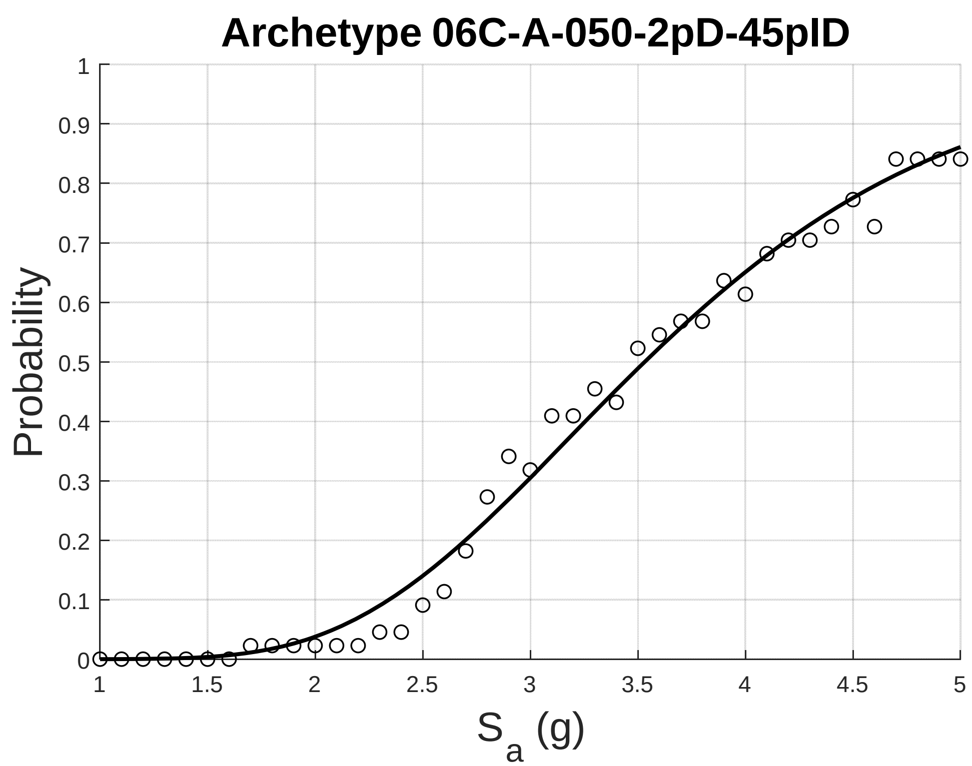


Figure G. 12. Archetype 06 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

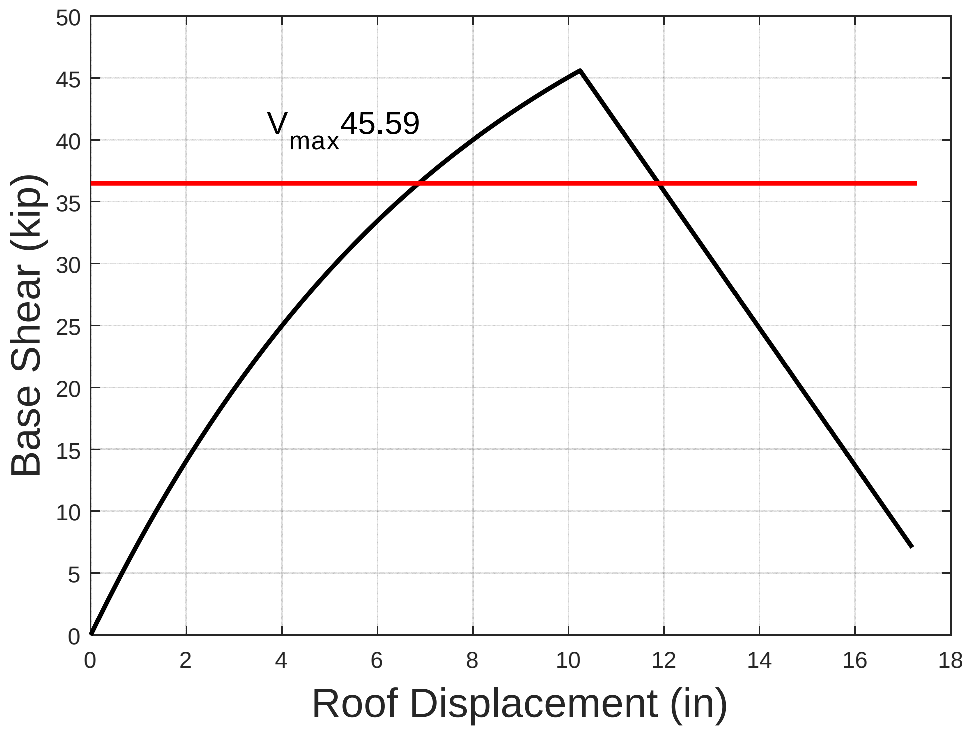


Figure G. 13. Archetype 13, R=3, static pushover

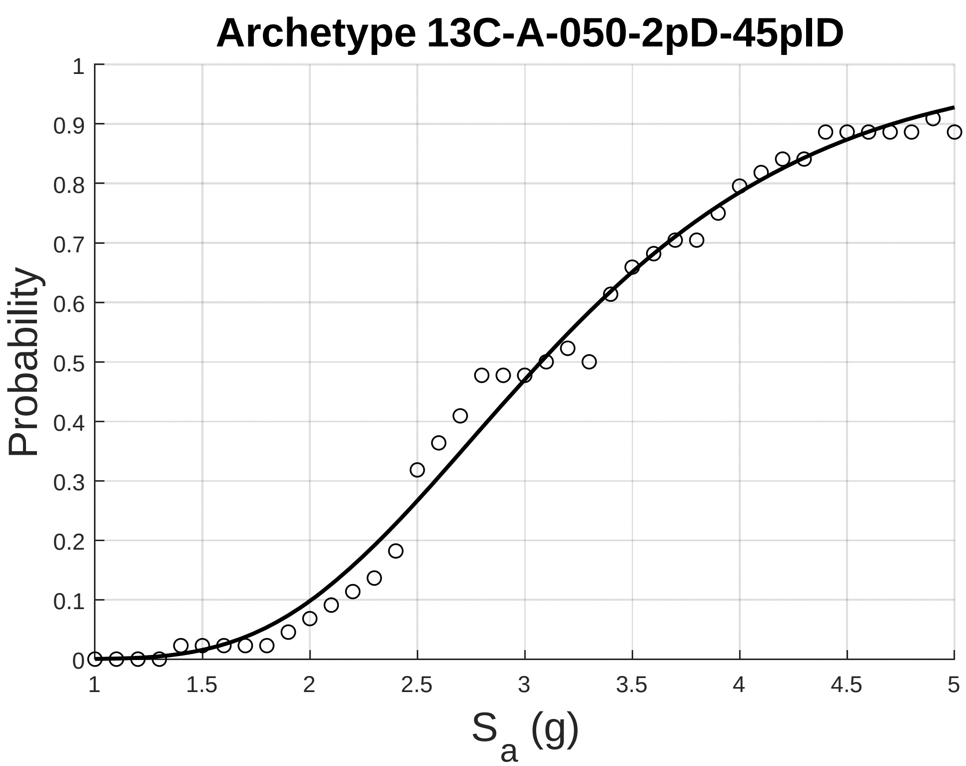


Figure G. 14. Archetype 13 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

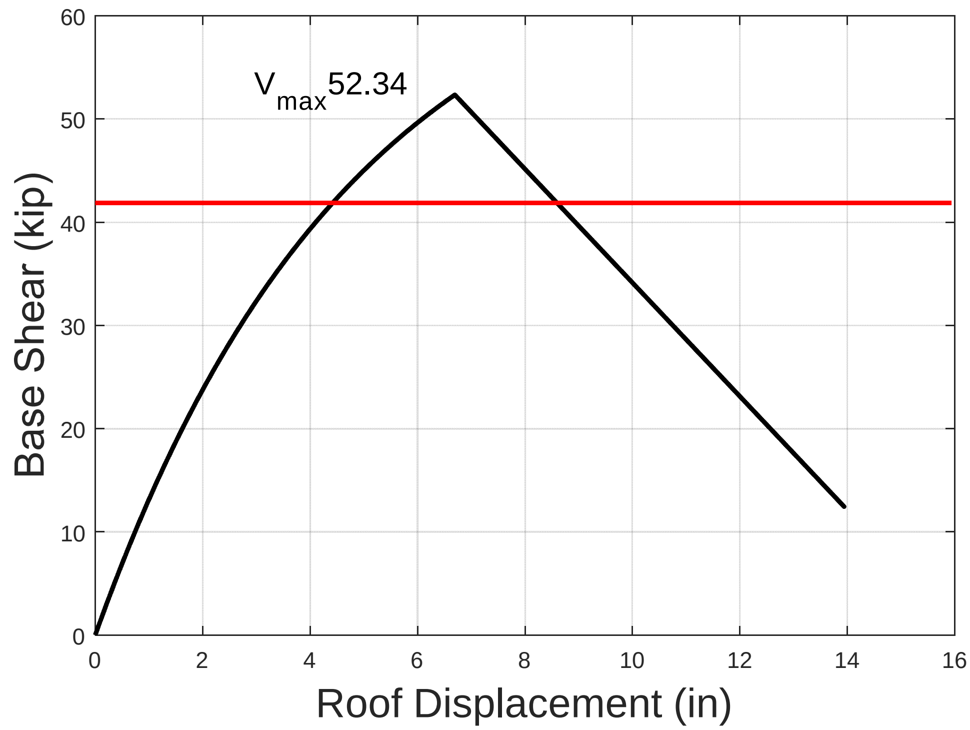


Figure G. 15. Archetype 14, R=3, static pushover

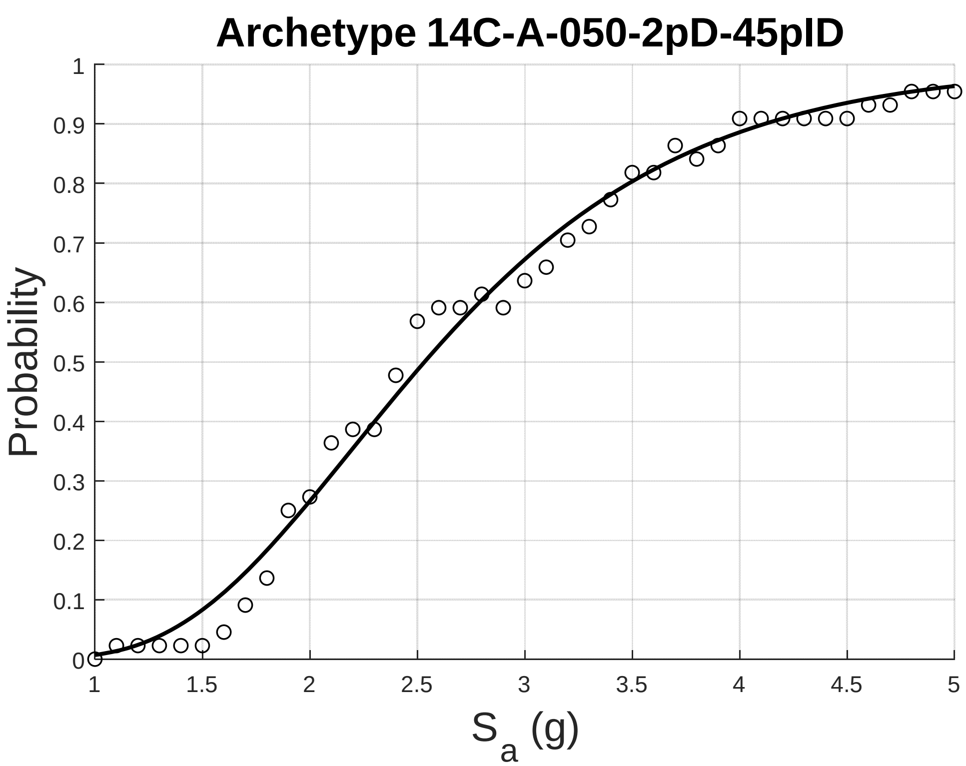


Figure G. 16. Archetype 14 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

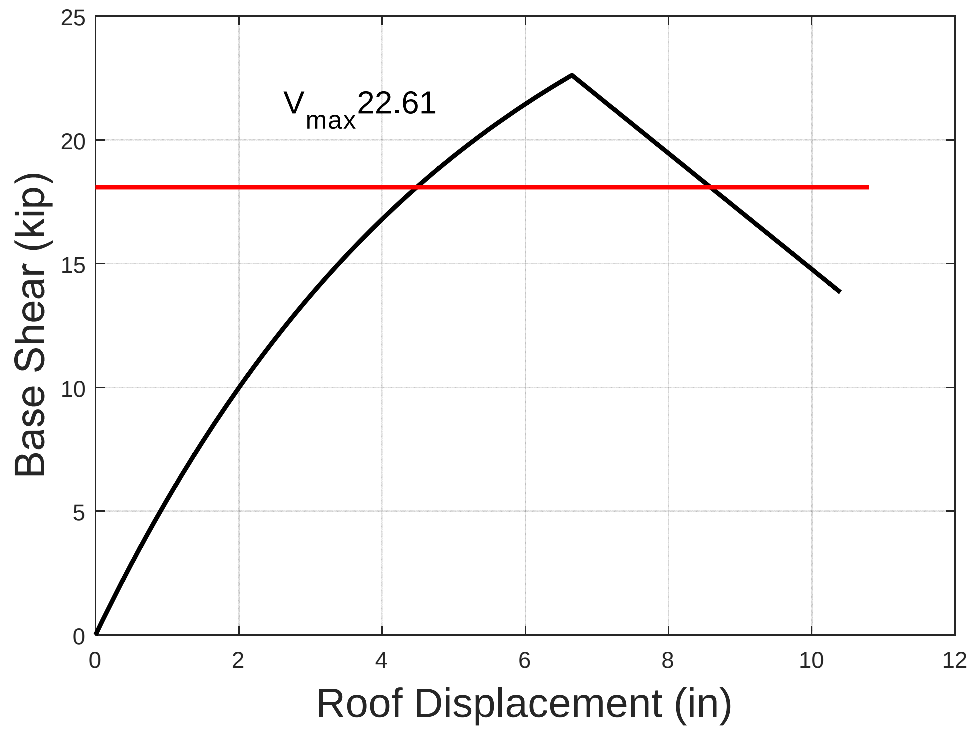


Figure G. 17. Archetype 15, R=3, static pushover

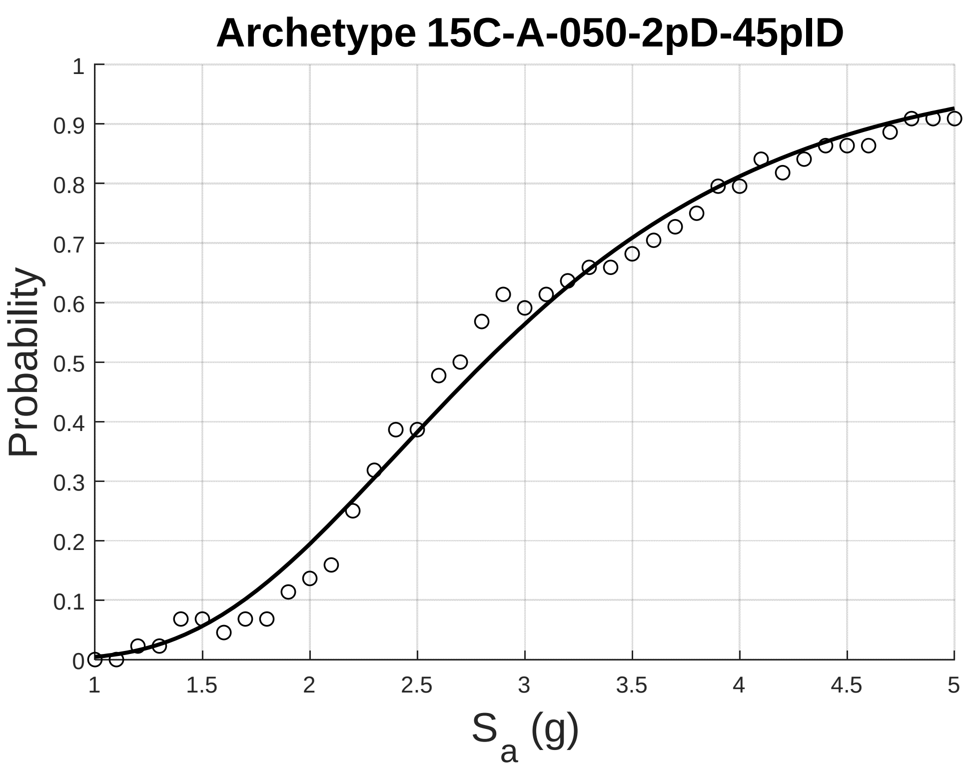


Figure G. 18. Archetype 15 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

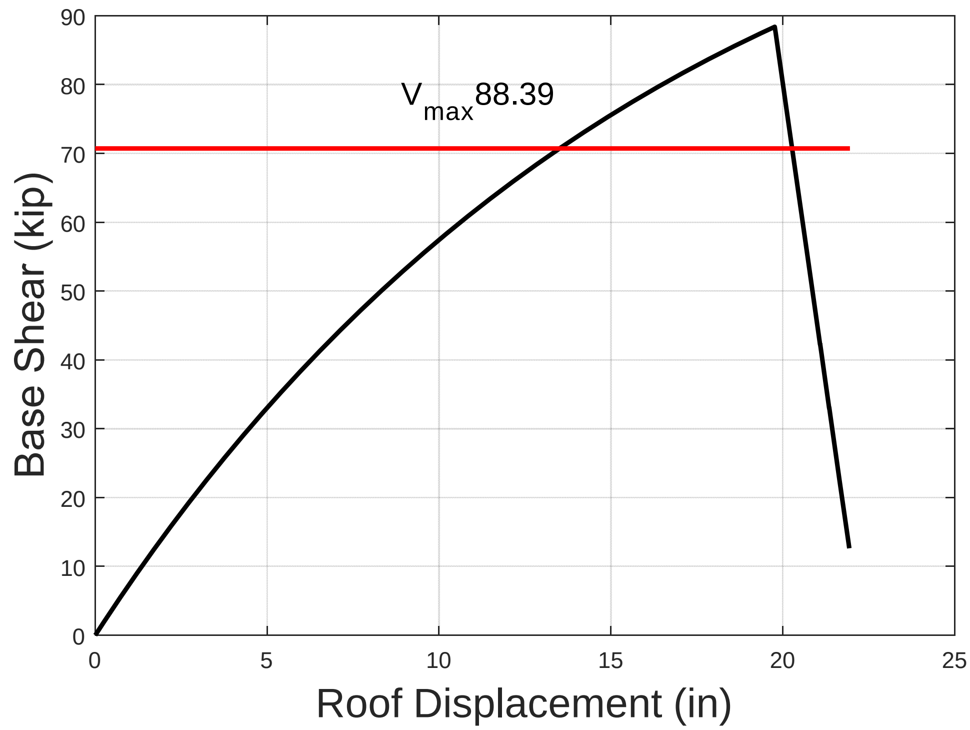


Figure G. 19. Archetype 16, R=3, static pushover

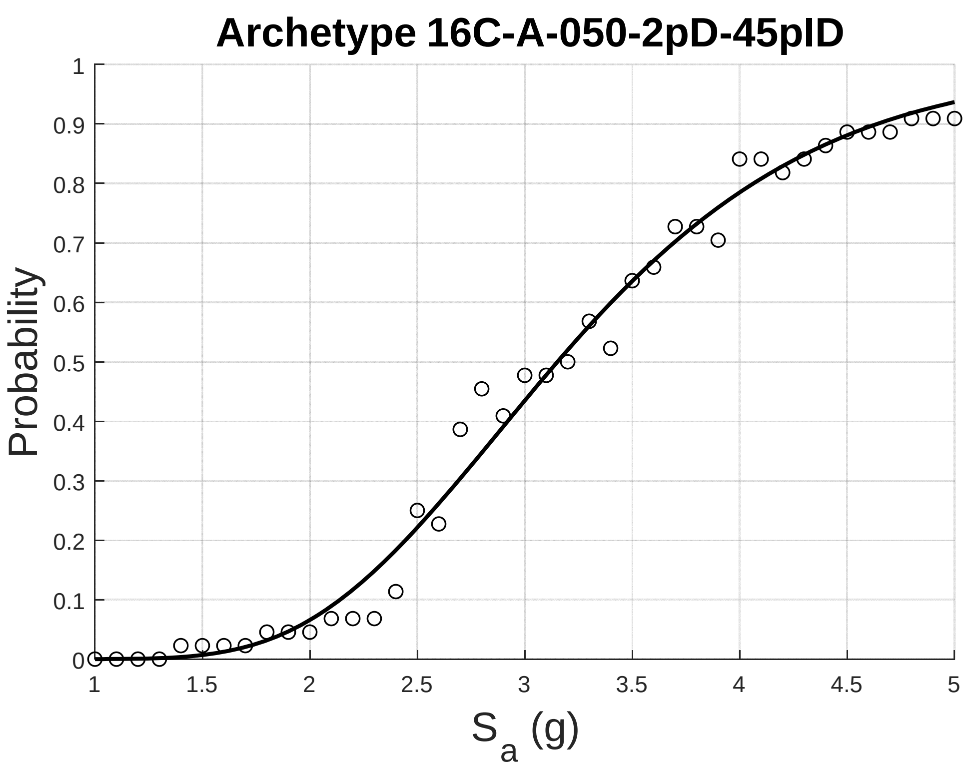


Figure G. 20. Archetype 16 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

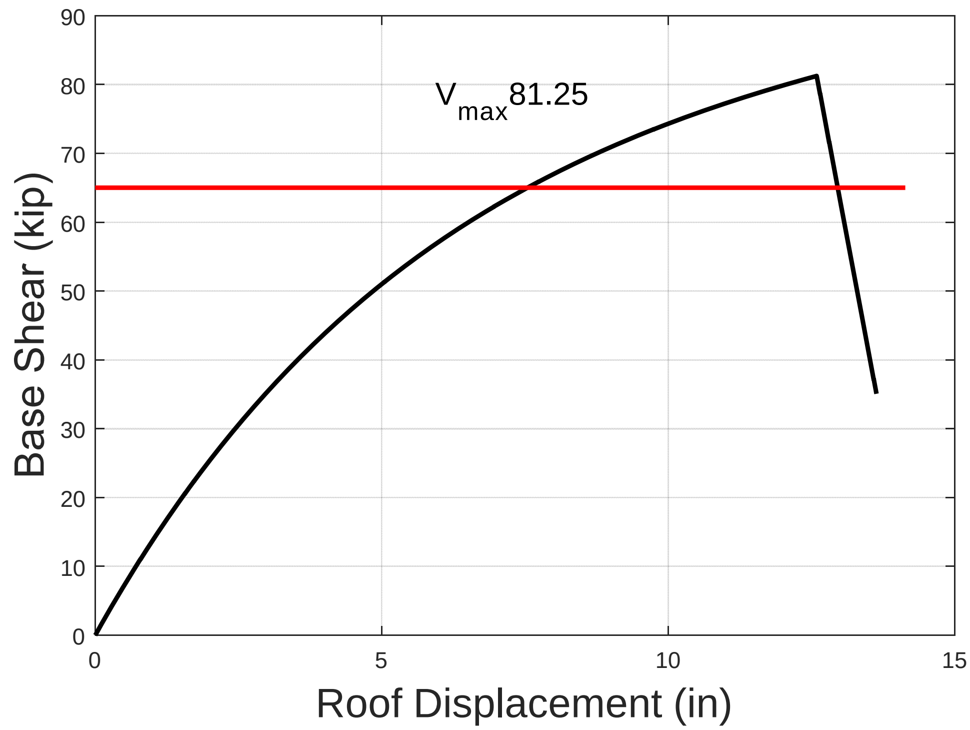


Figure G. 21. Archetype 17, R=3, static pushover

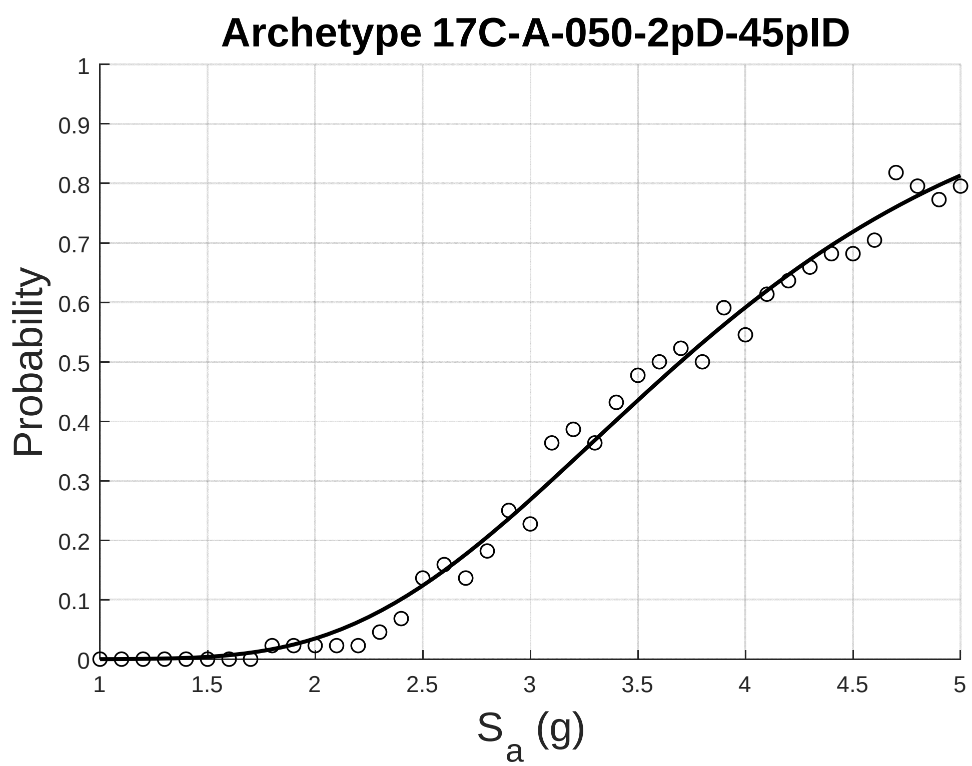


Figure G. 22. Archetype 17 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

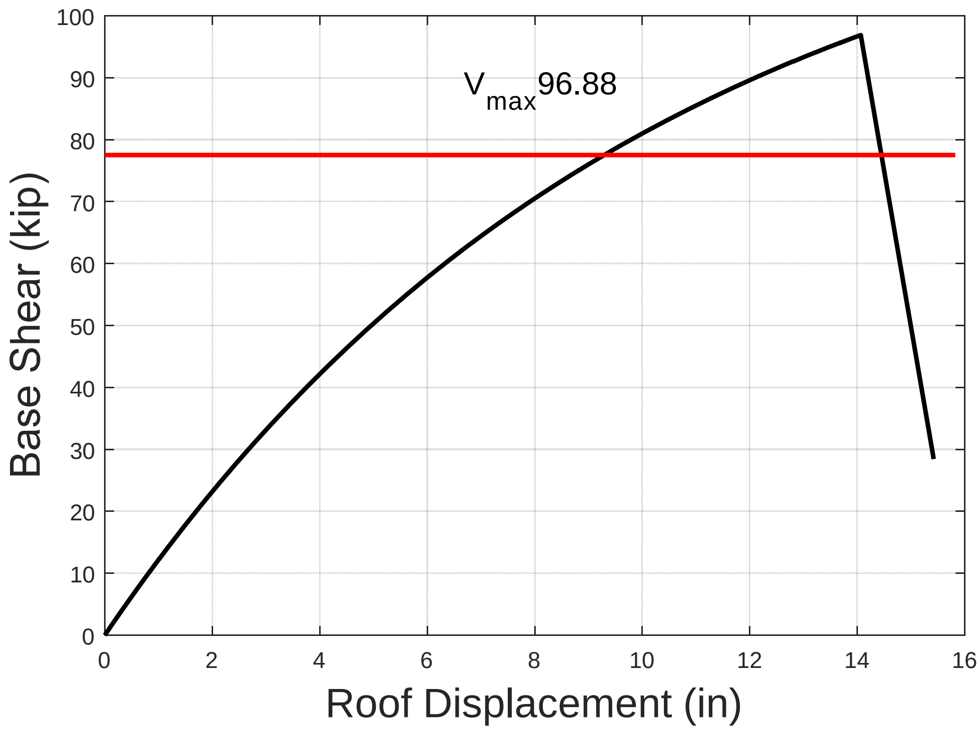


Figure G. 23. Archetype 18, R=3, static pushover

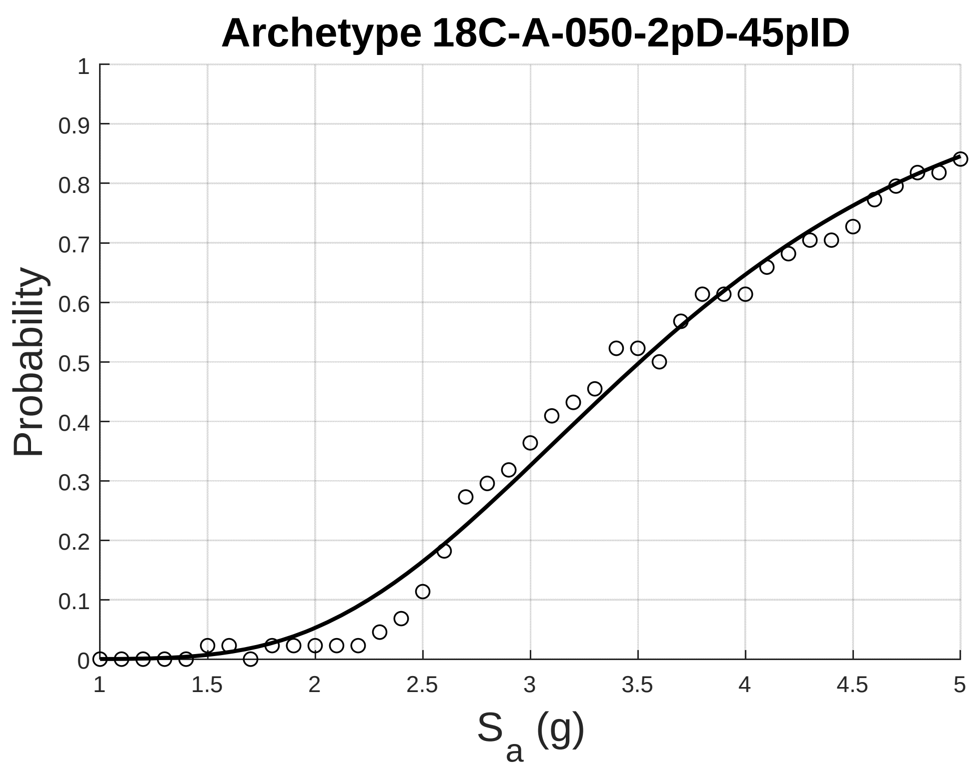


Figure G. 24. Archetype 18 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

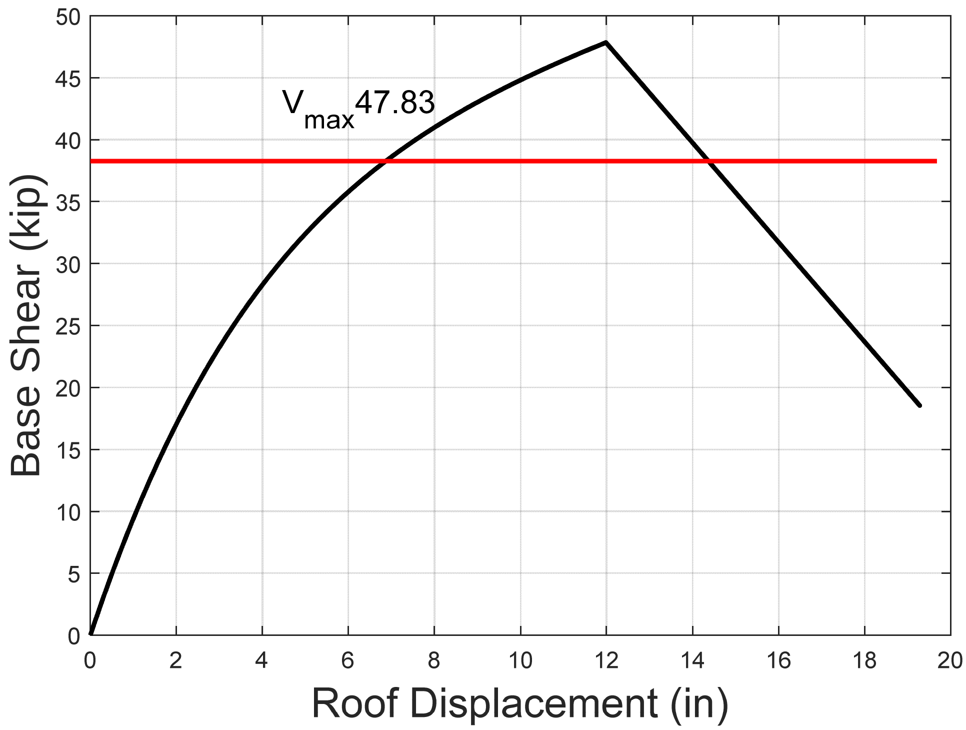


Figure G. 25. Archetype 25, R=3, static pushover

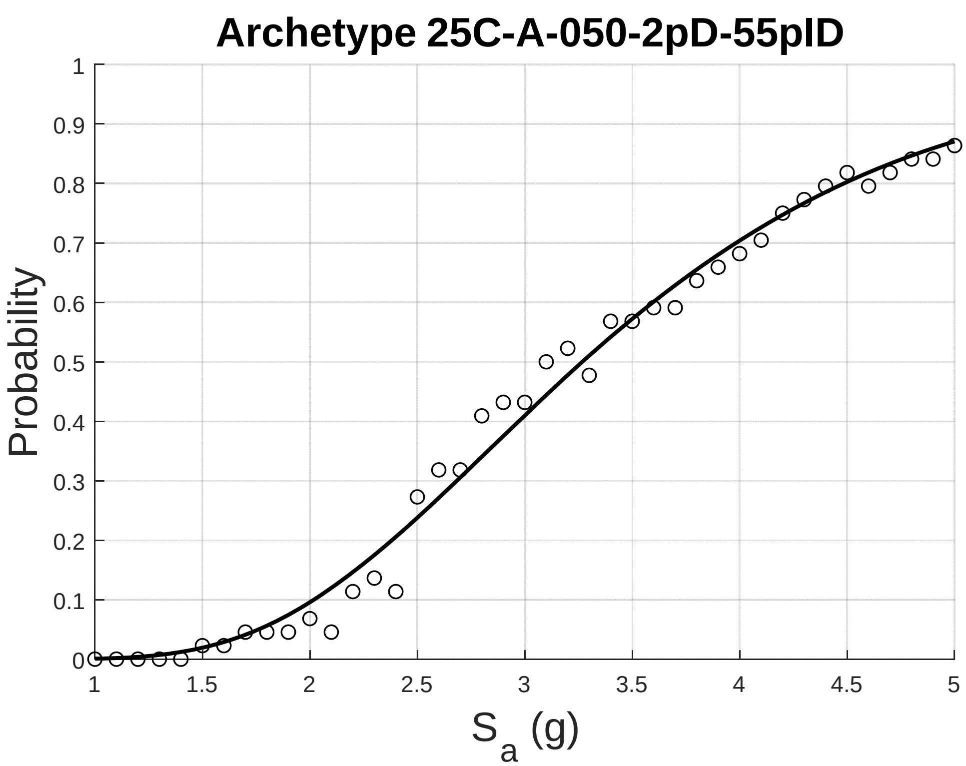


Figure G. 26. Archetype 25 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

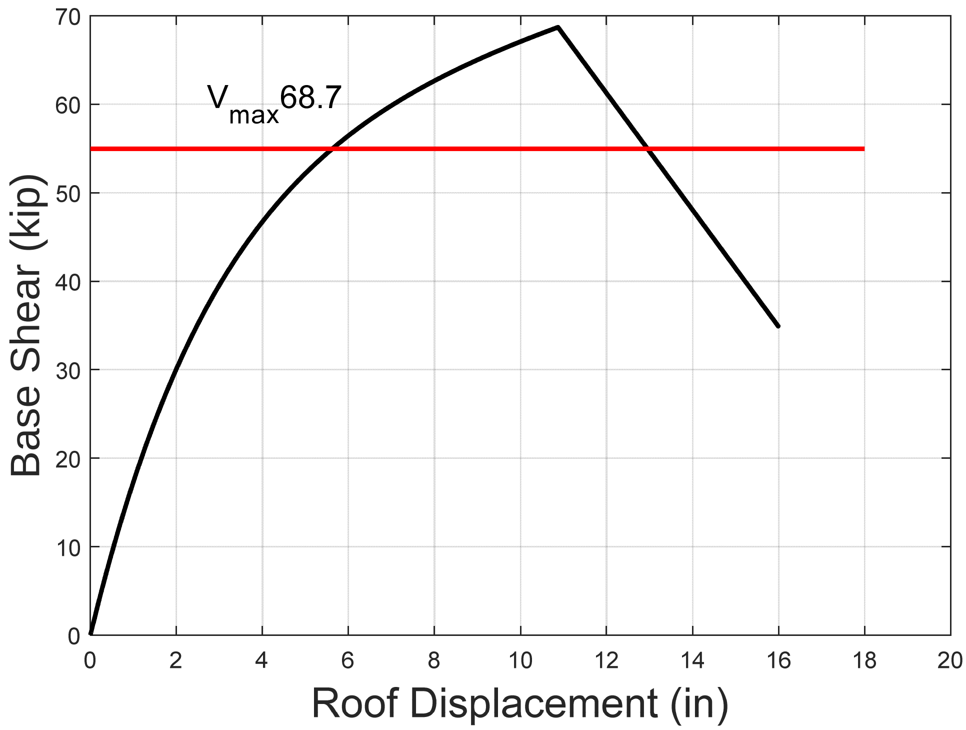


Figure G. 27. Archetype 26, R=3, static pushover

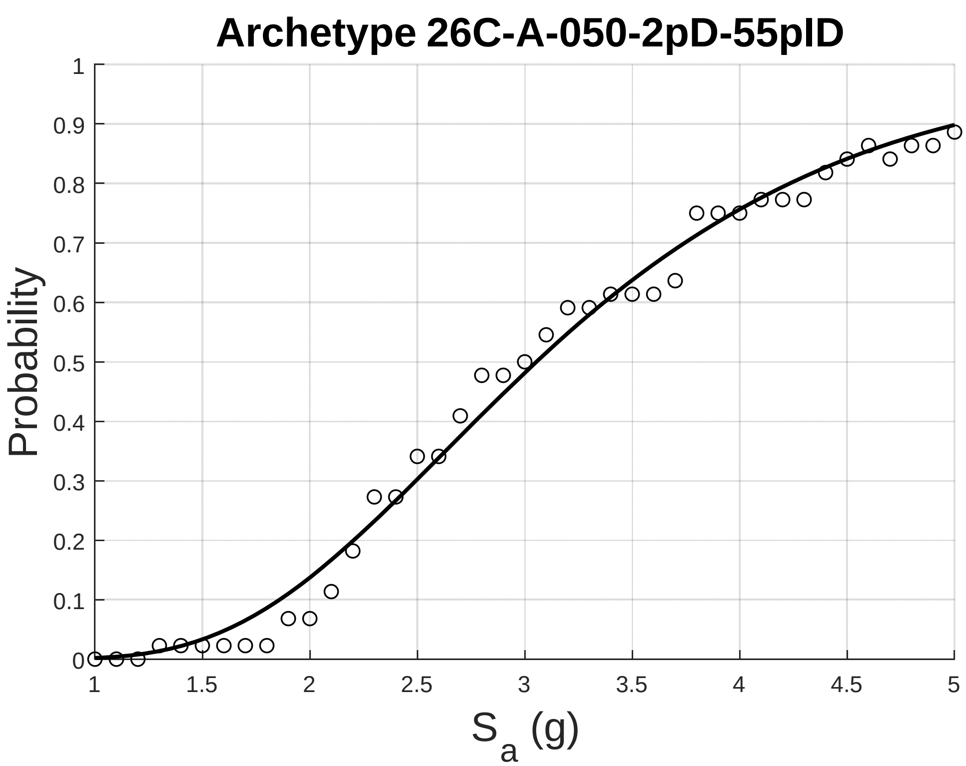


Figure G. 28. Archetype 26 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

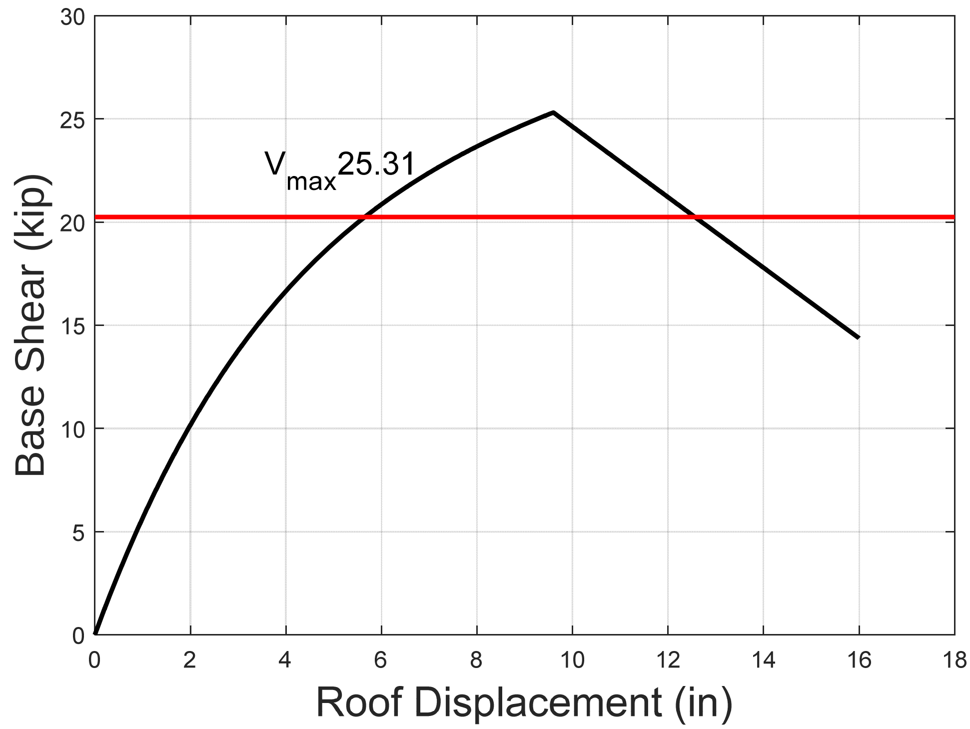


Figure G. 29. Archetype 27, R=3, static pushover

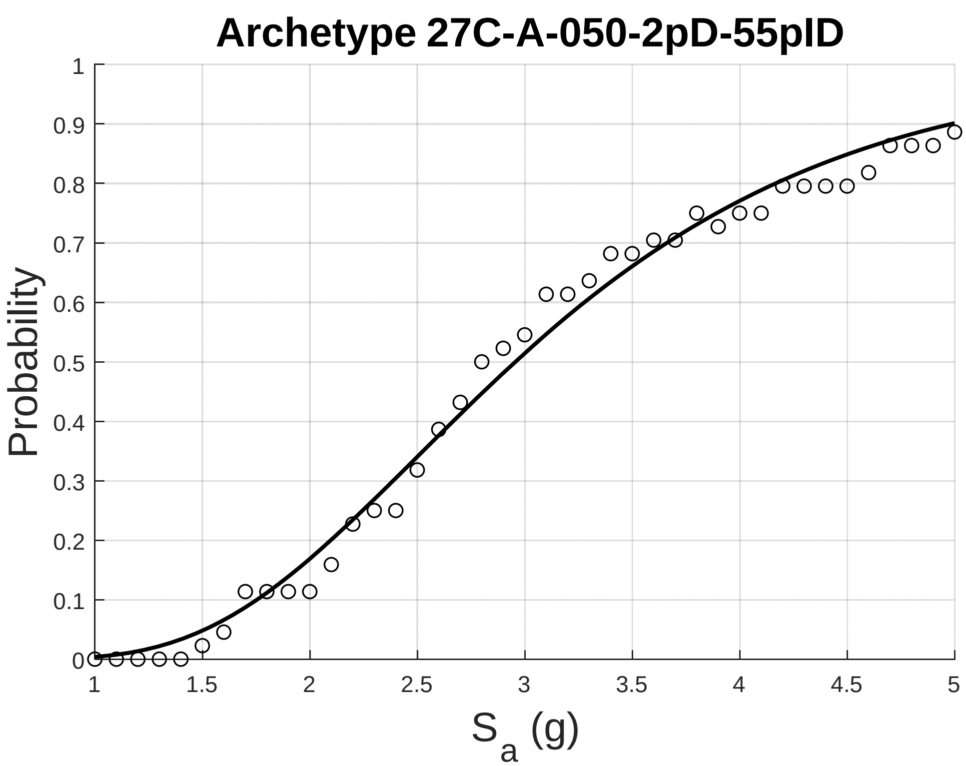


Figure G. 30. Archetype 27 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

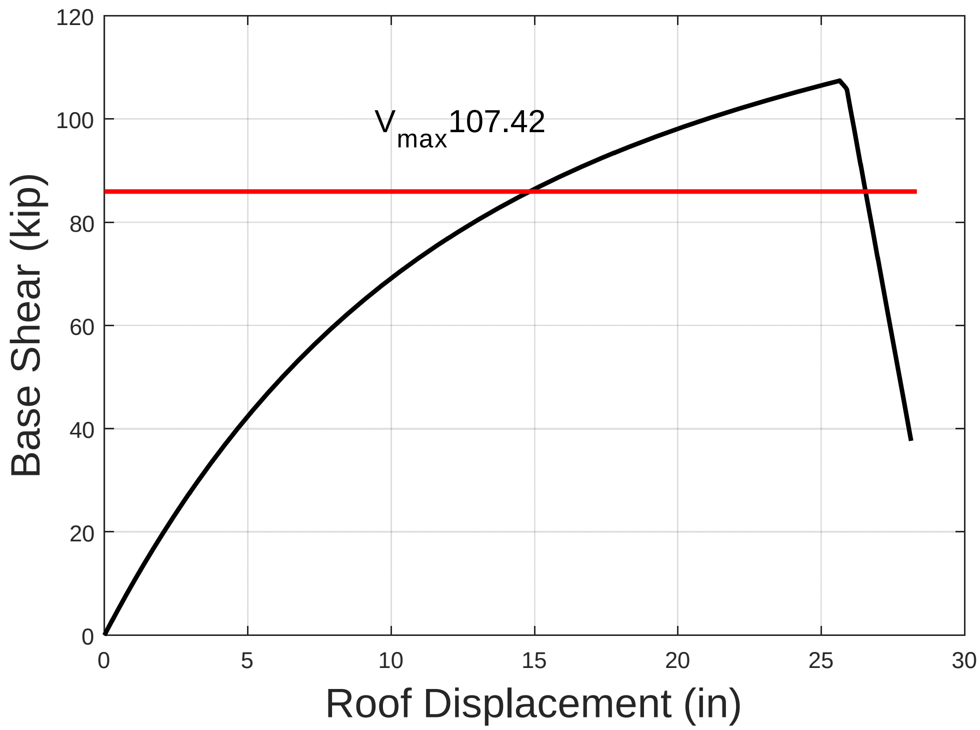


Figure G. 31. Archetype 28, R=3, static pushover

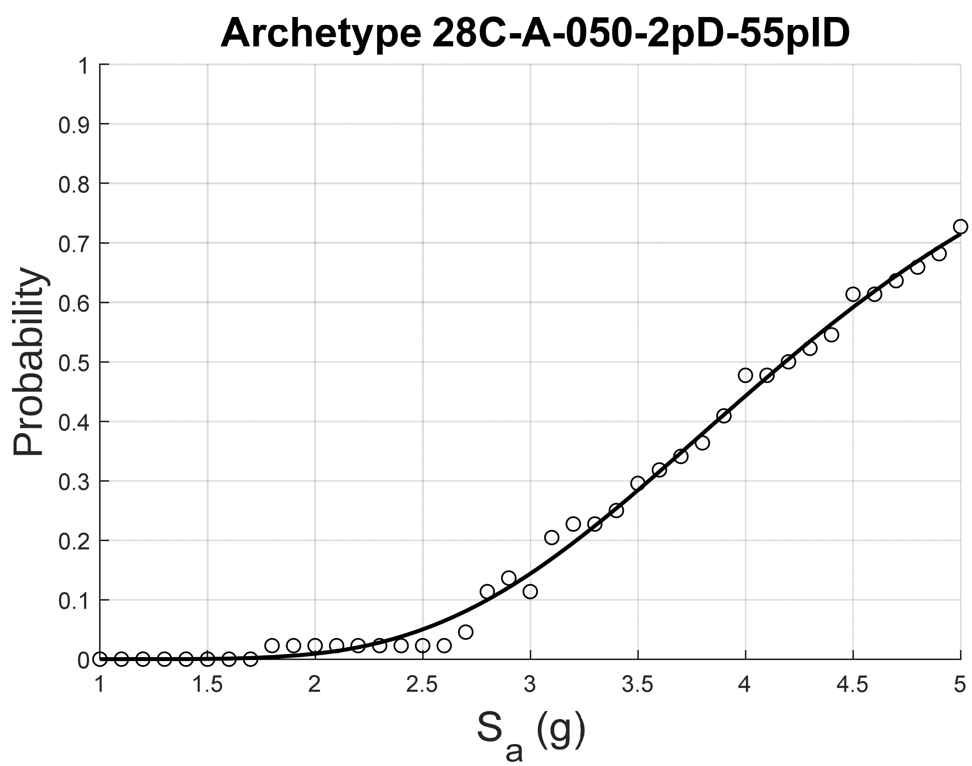


Figure G. 32. Archetype 28 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

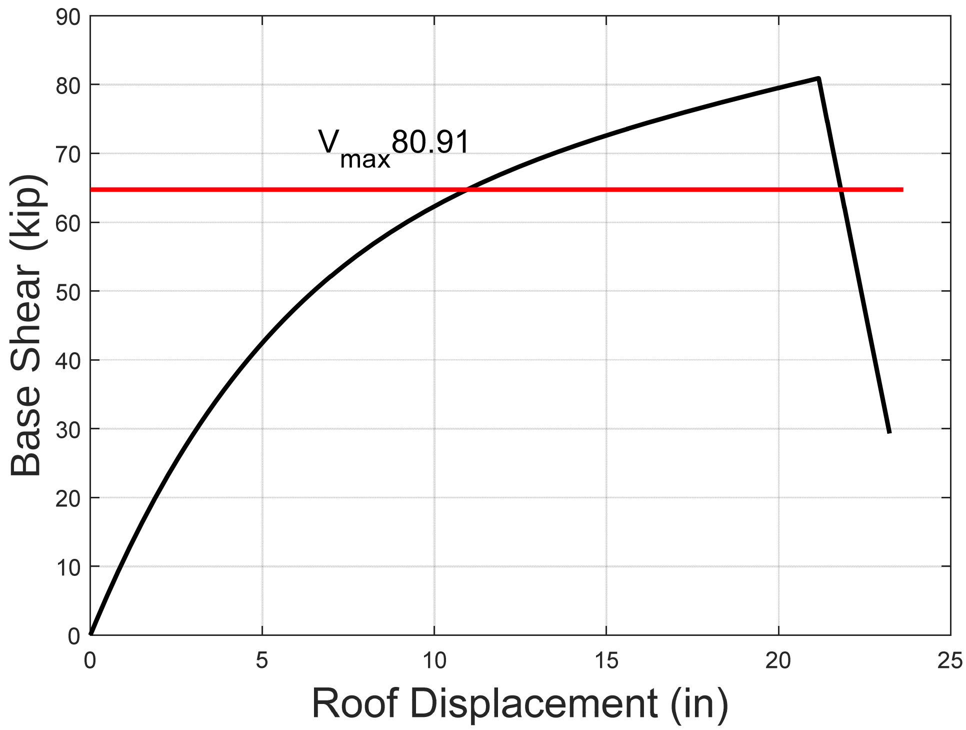


Figure G. 33. Archetype 29, R=3, static pushover

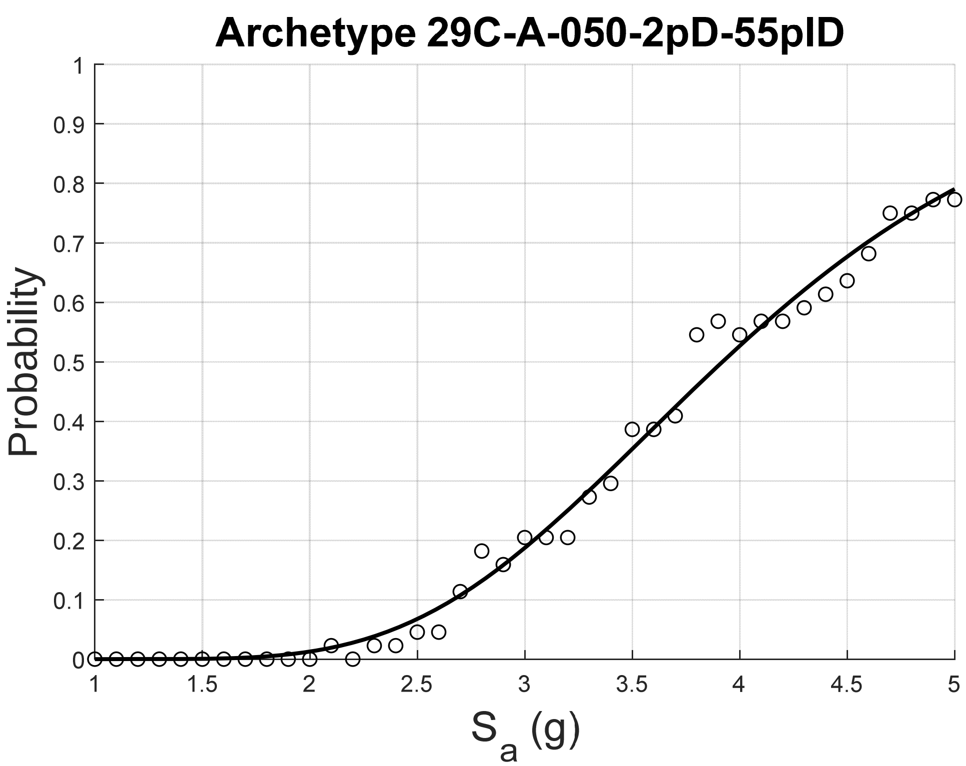


Figure G. 34. Archetype 29 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

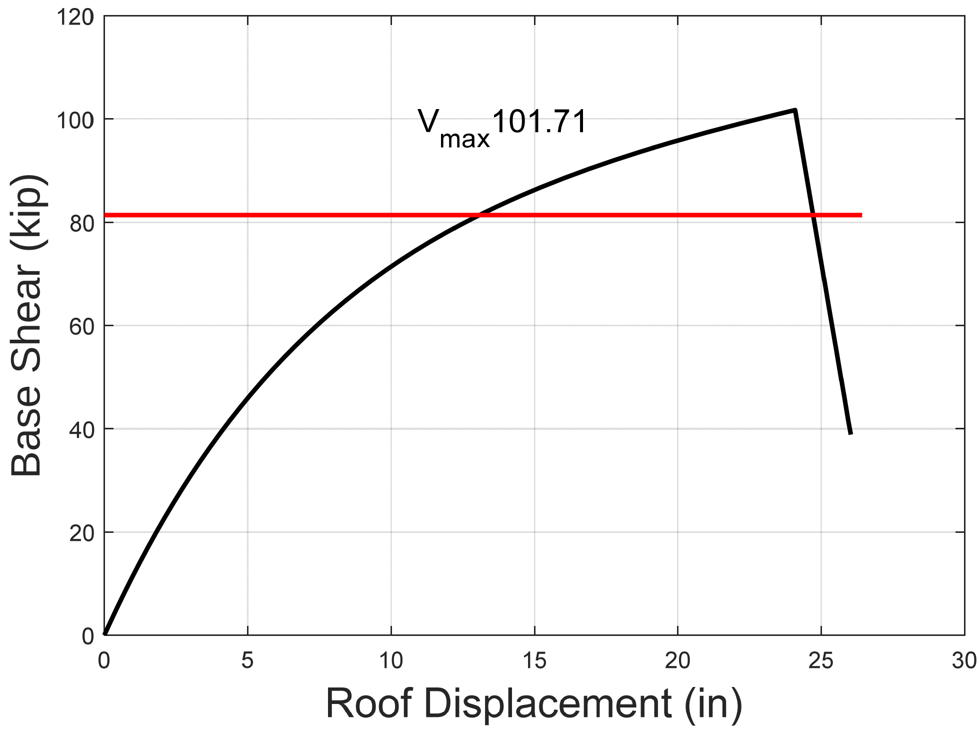


Figure G. 35. Archetype 30, R=3, static pushover

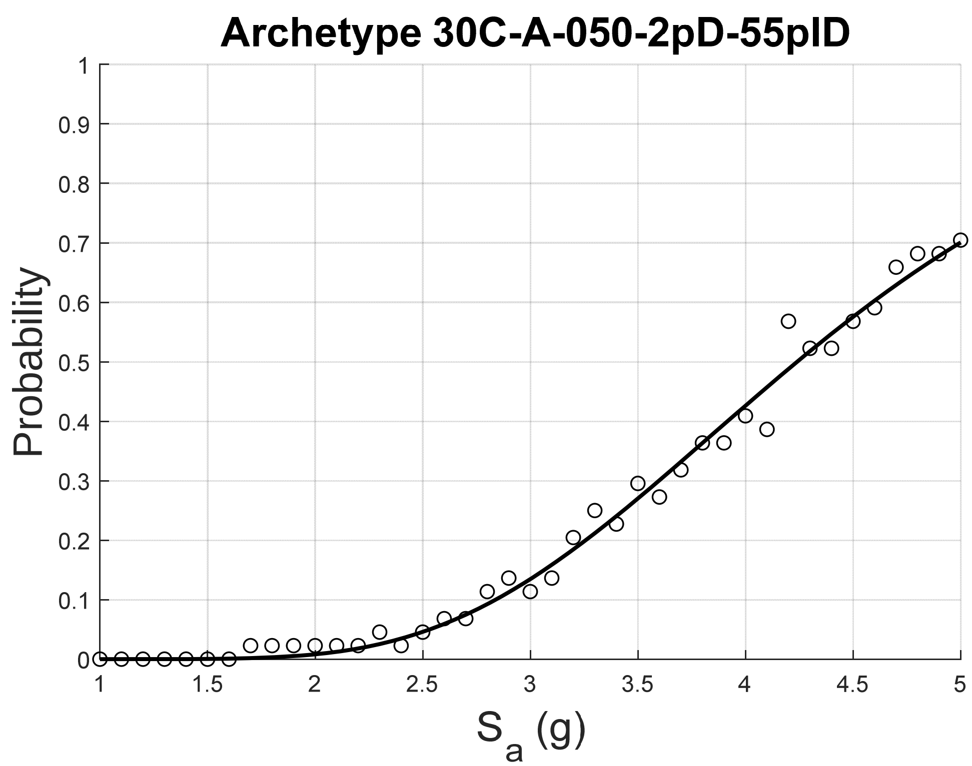


Figure G. 36. Archetype 30 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

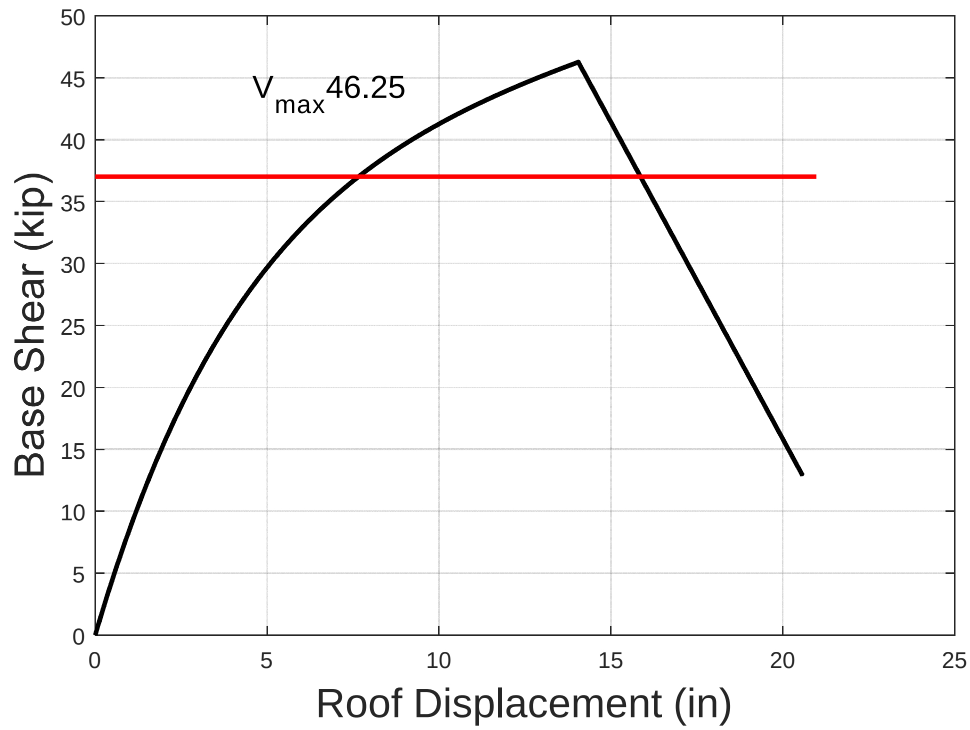


Figure G. 37. Archetype 37, R=3, static pushover

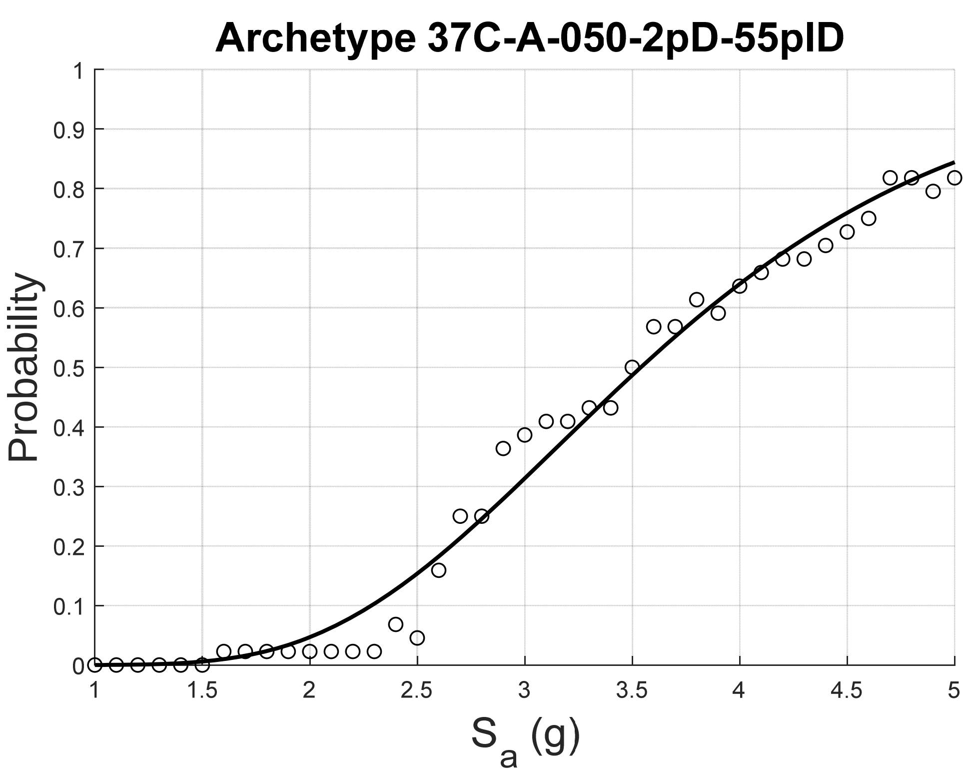


Figure G. 38. Archetype 37 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

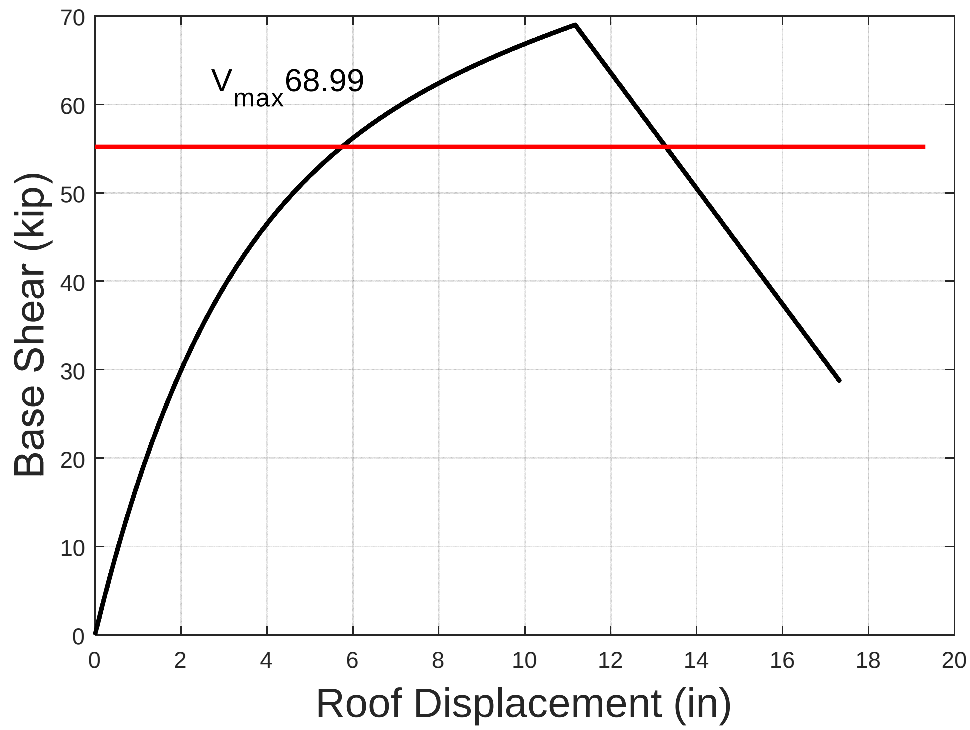


Figure G. 39. Archetype 38, R=3, static pushover

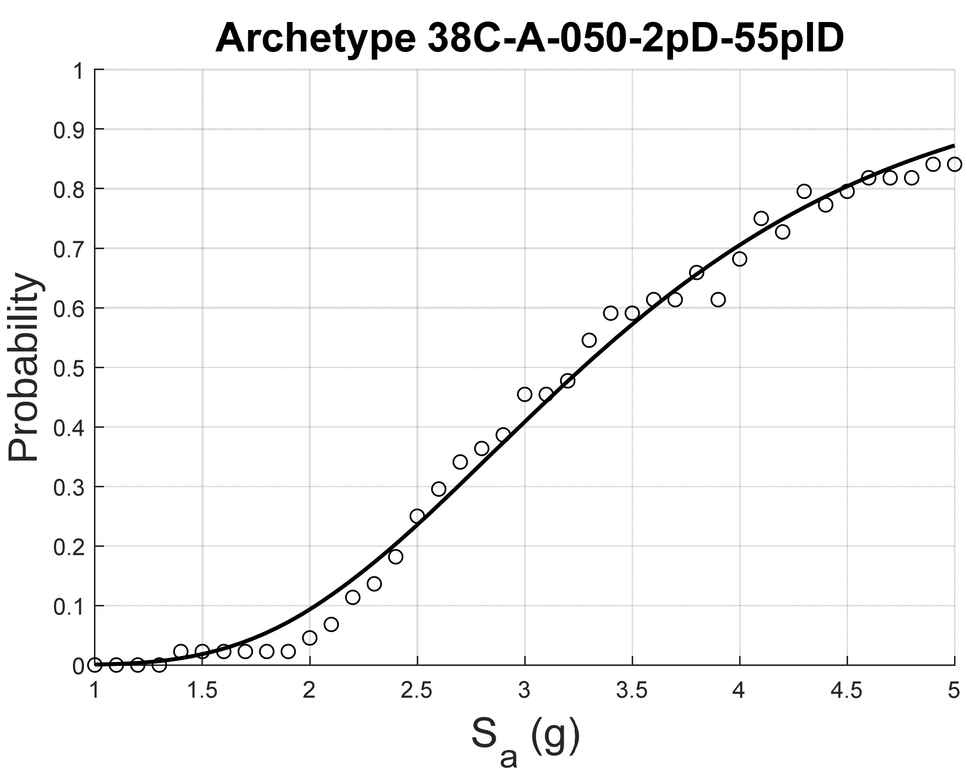


Figure G. 40. Archetype 38 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

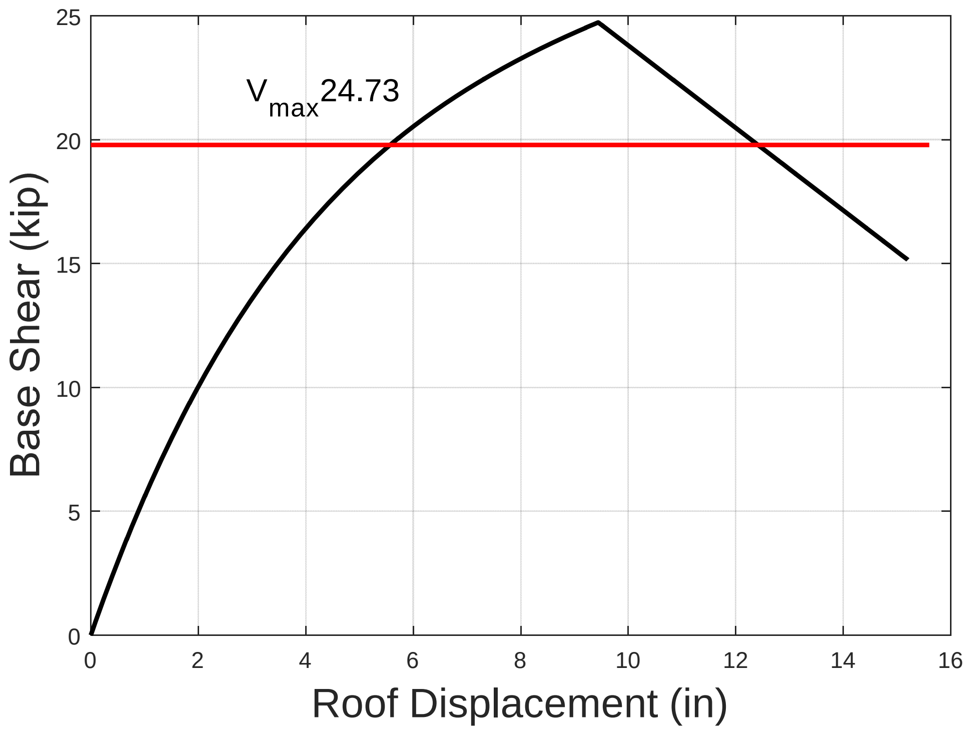


Figure G. 41. Archetype 39, R=3, static pushover

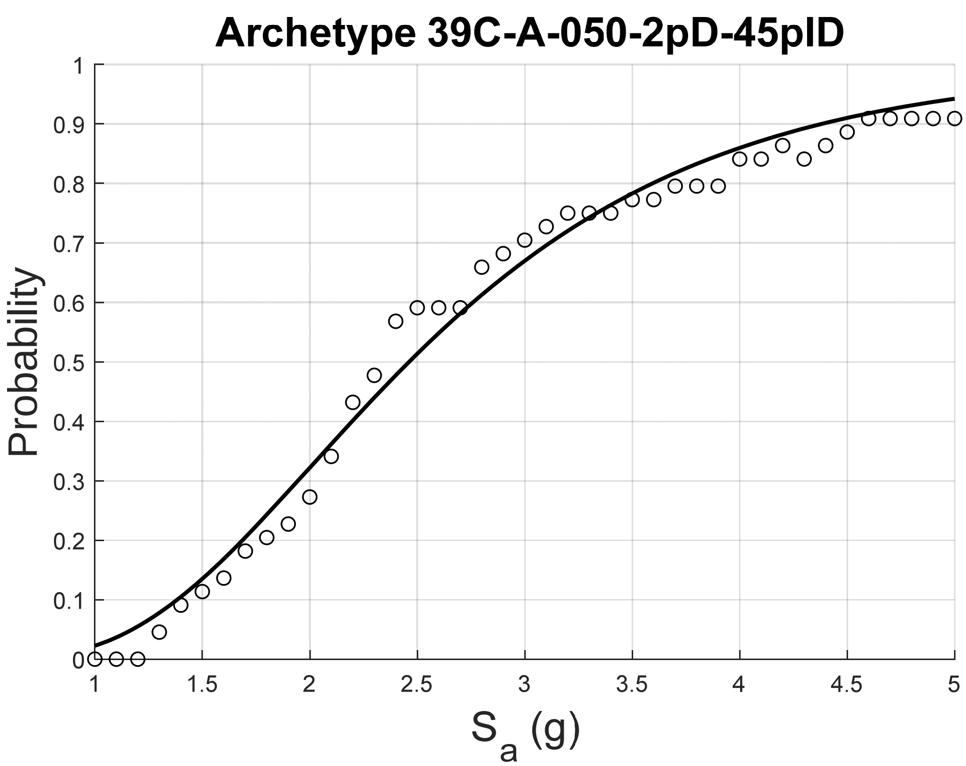


Figure G. 42. Archetype 39 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

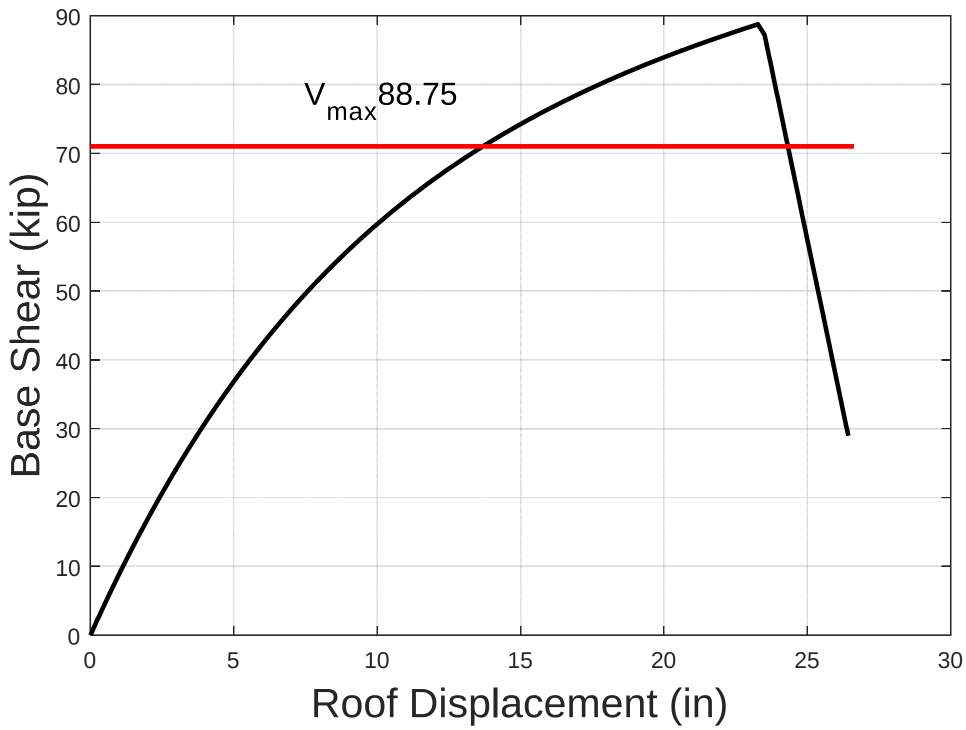


Figure G. 43. Archetype 40, R=3, static pushover

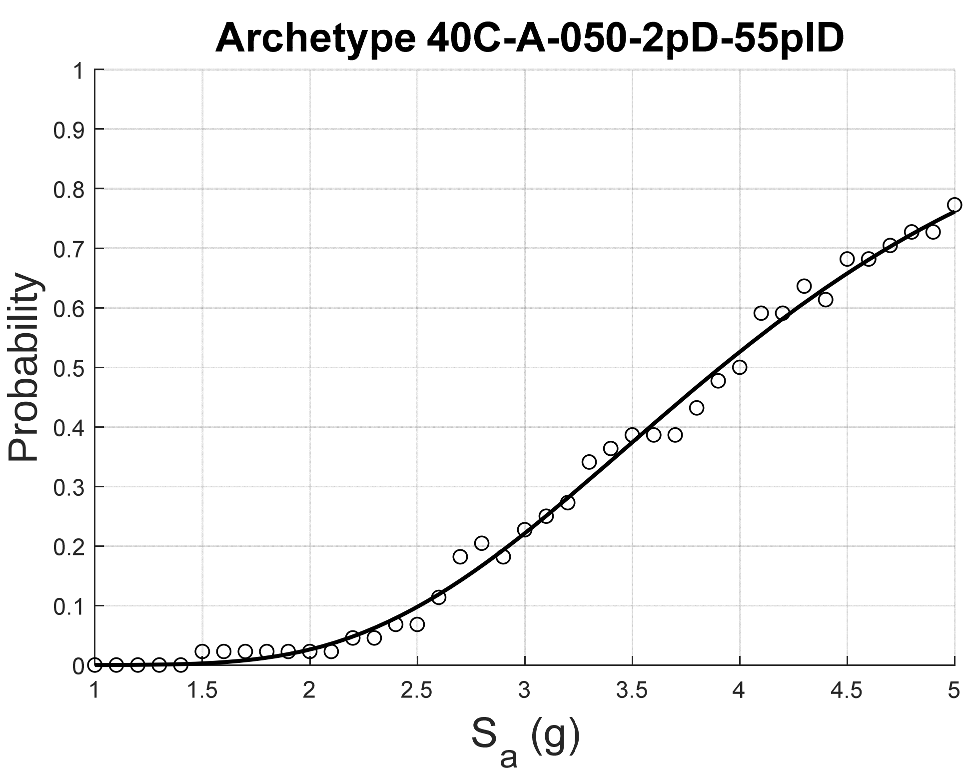


Figure G. 44. Archetype 40 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

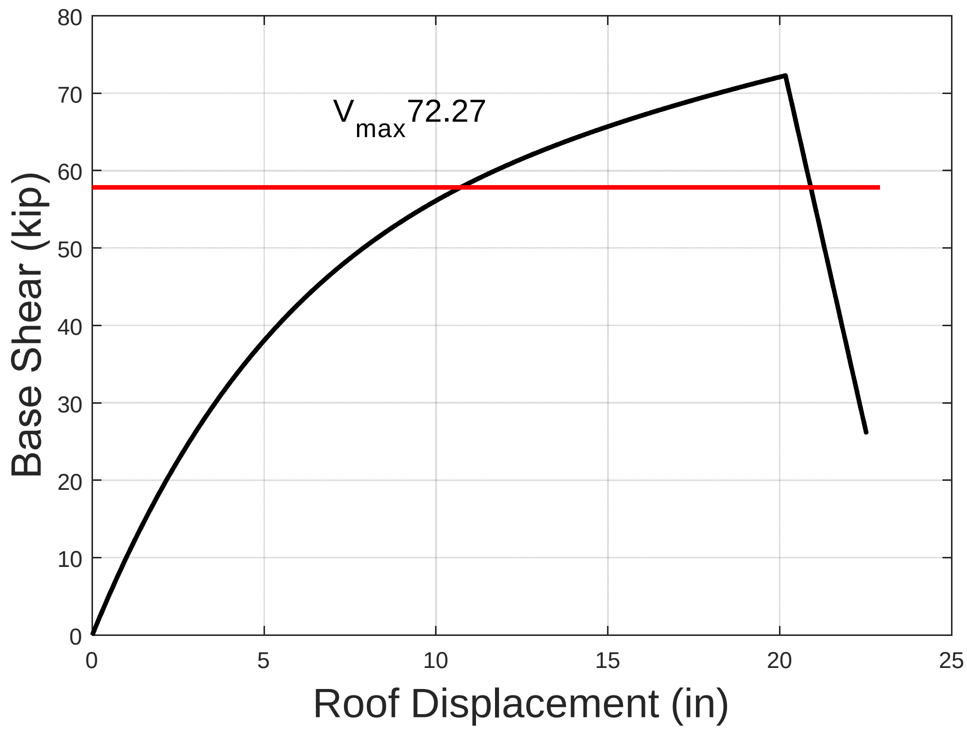


Figure G. 45. Archetype 41, R=3, static pushover

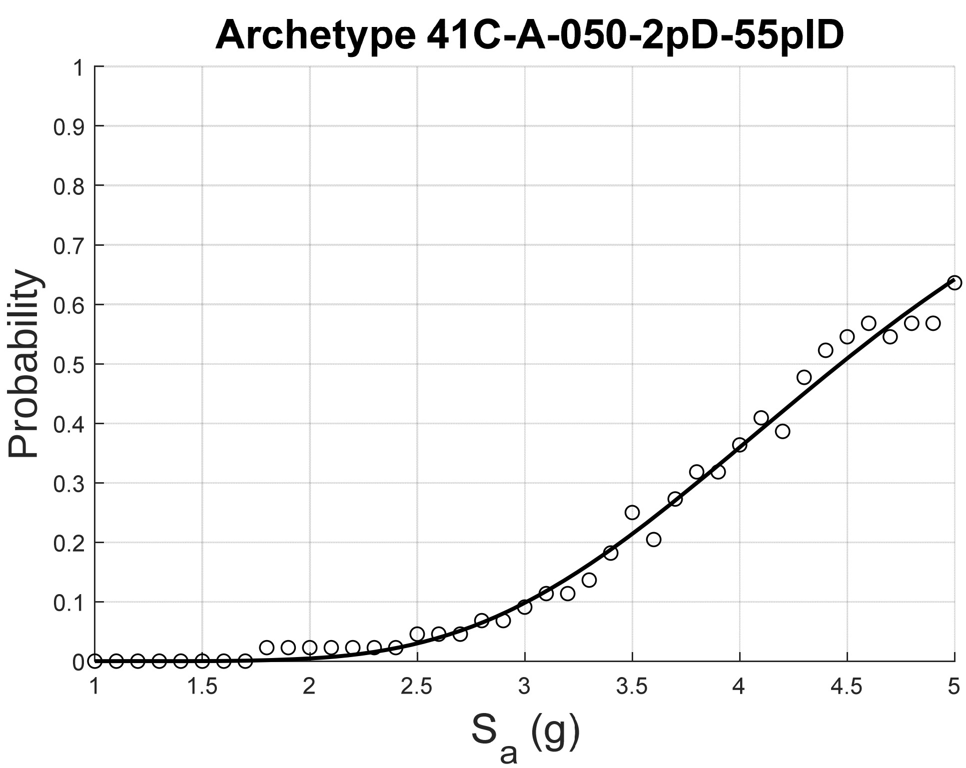


Figure G. 46. Archetype 41 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

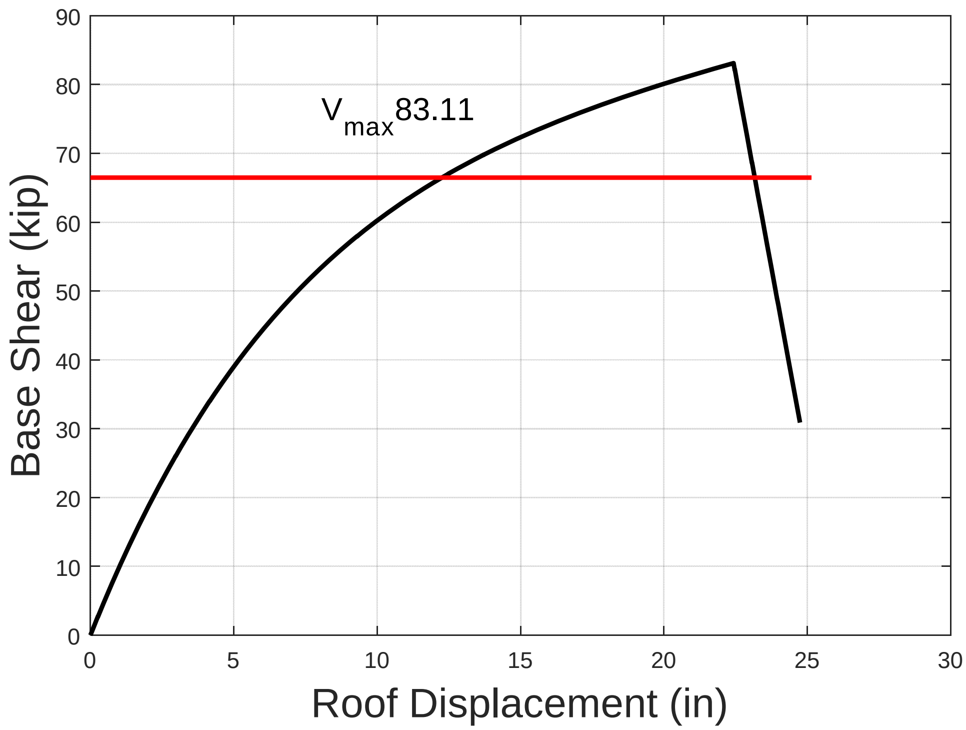


Figure G. 47. Archetype 42, R=3, static pushover

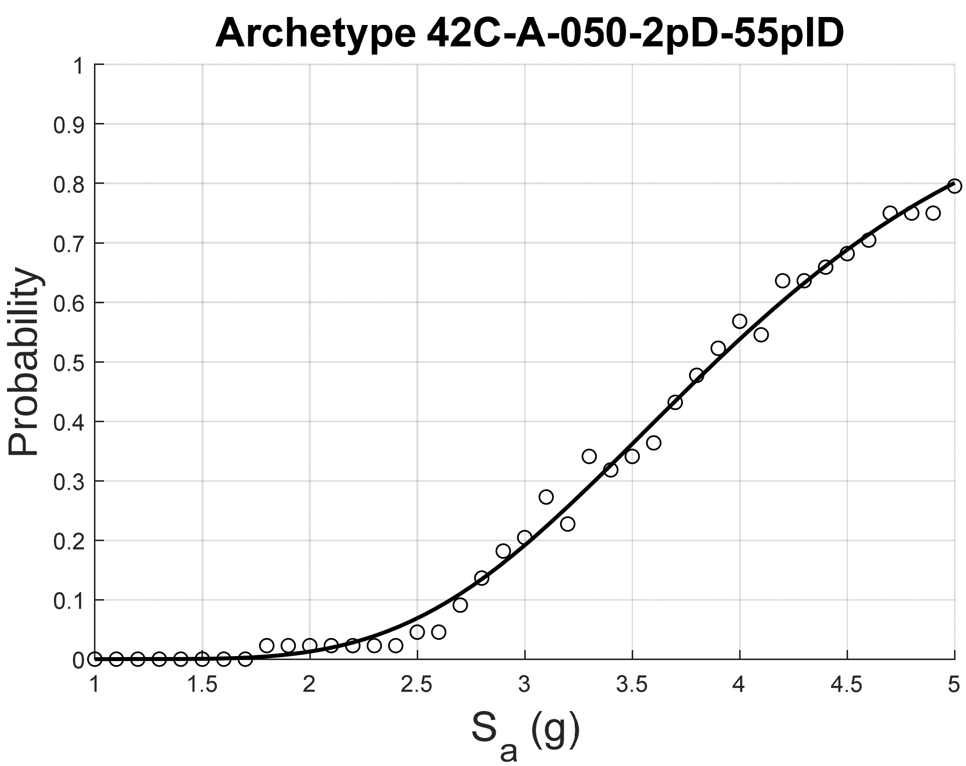


Figure G. 48. Archetype 42 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

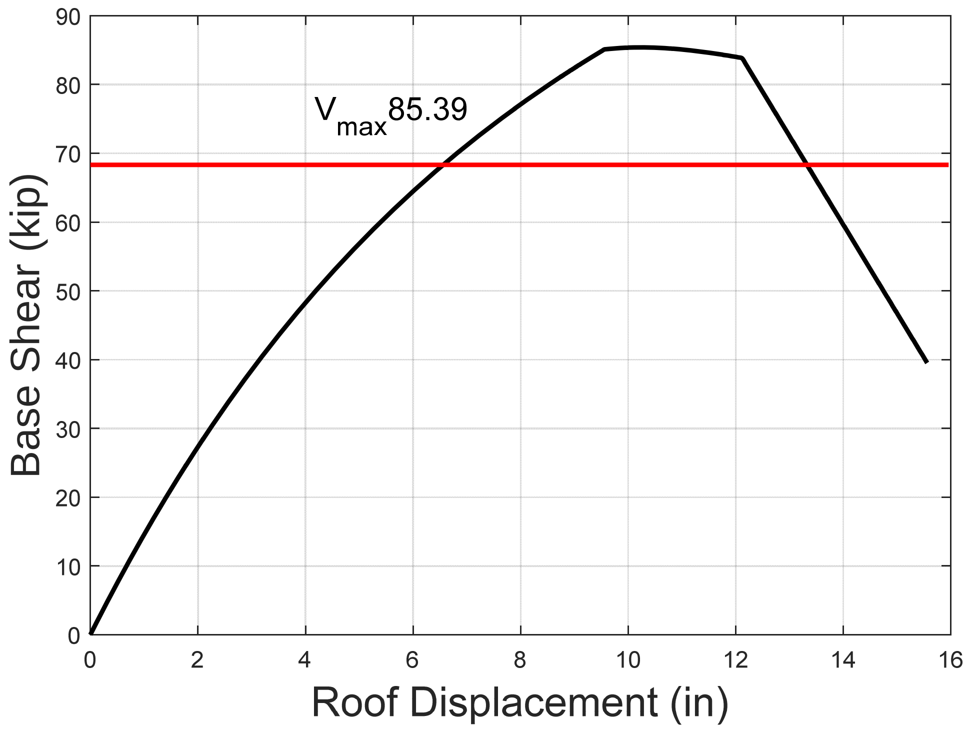


Figure G. 49. Archetype 49, R=3, static pushover

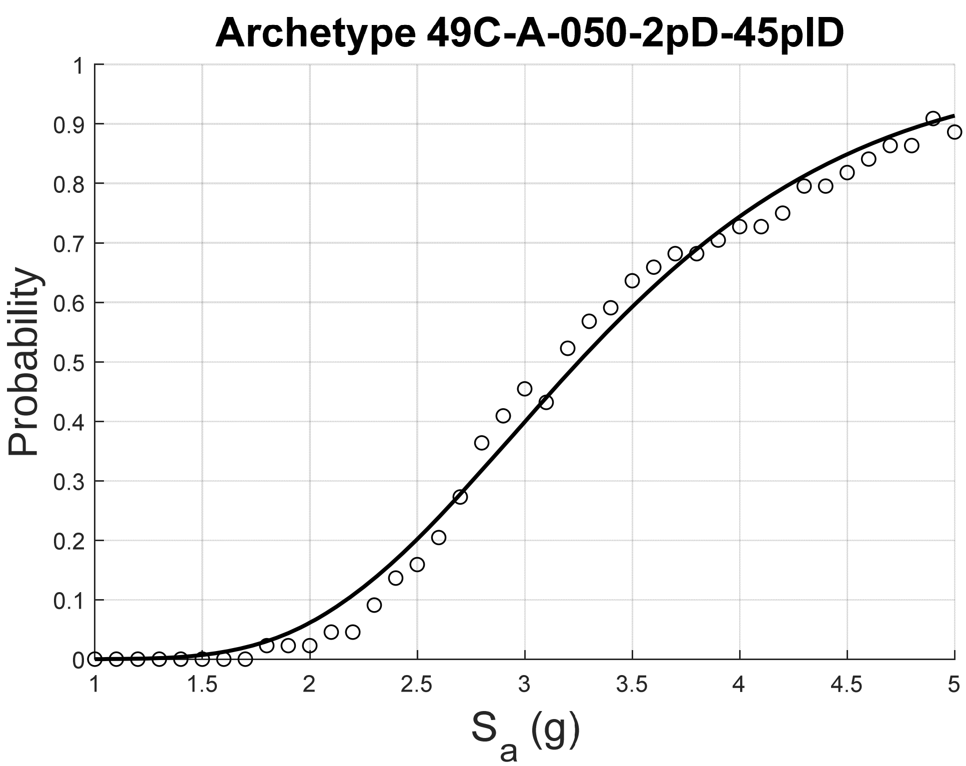


Figure G. 50. Archetype 49 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

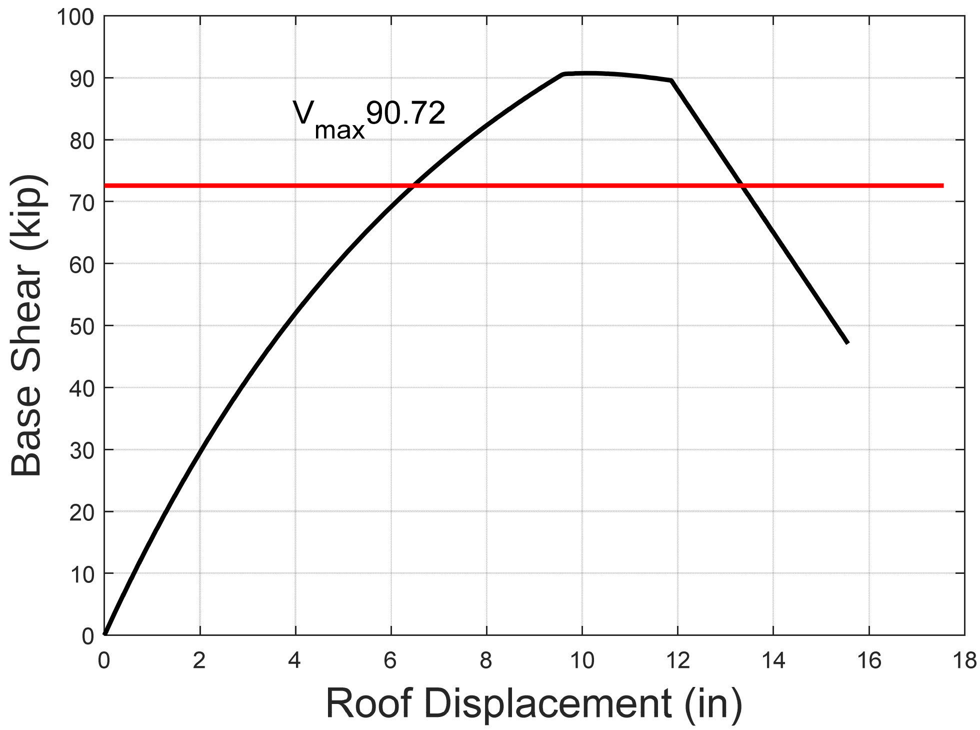


Figure G. 51. Archetype 50, R=3, static pushover

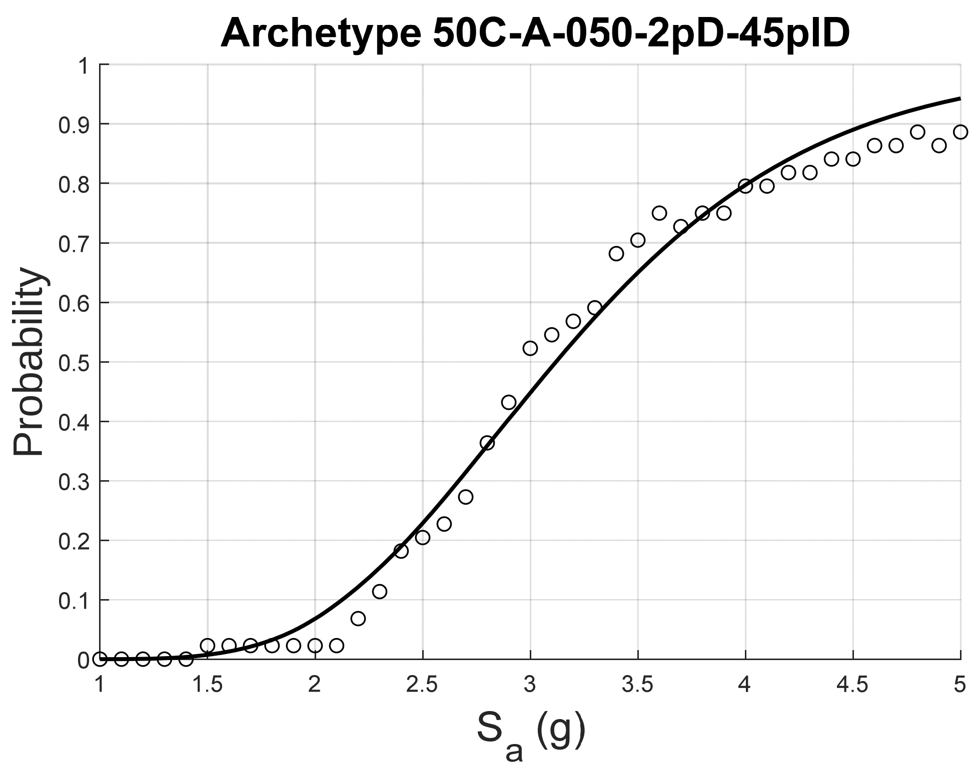


Figure G. 52. Archetype 50 collapse fragility, R=3, 2% damping, 4% interstory drift limit

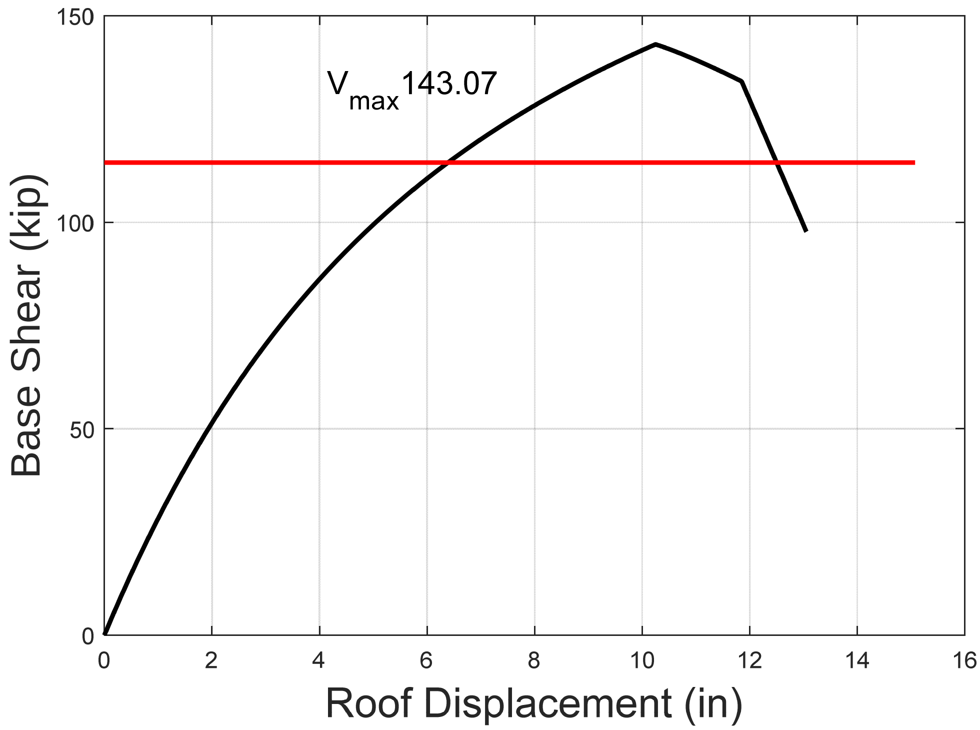


Figure G. 53. Archetype 51, R=3, static pushover

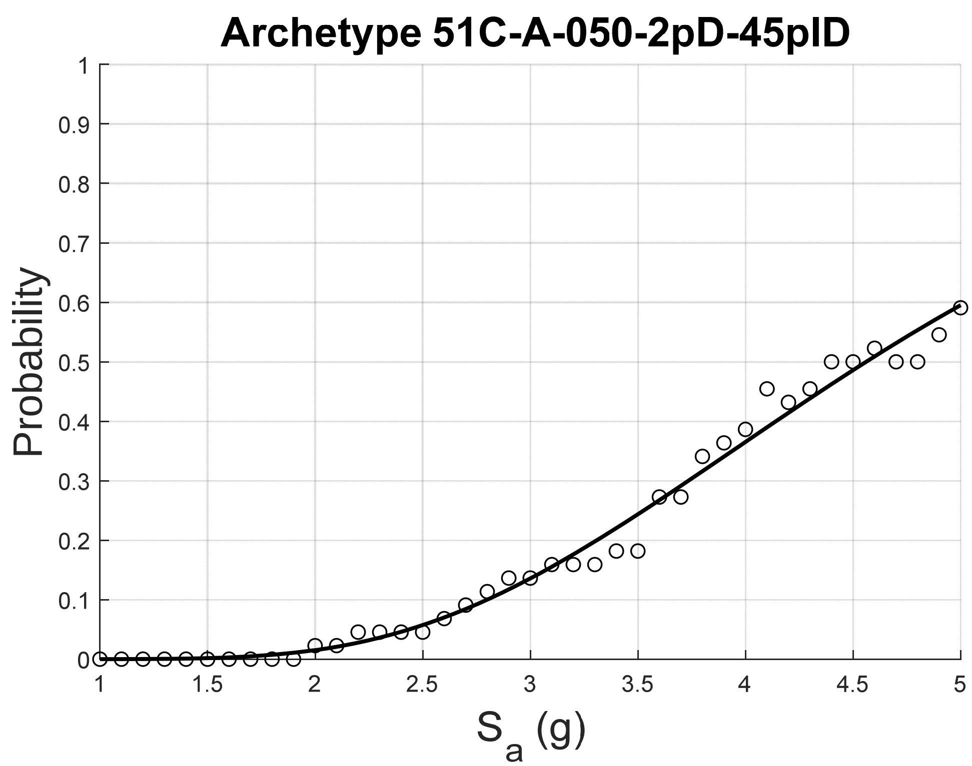


Figure G. 54. Archetype 51 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

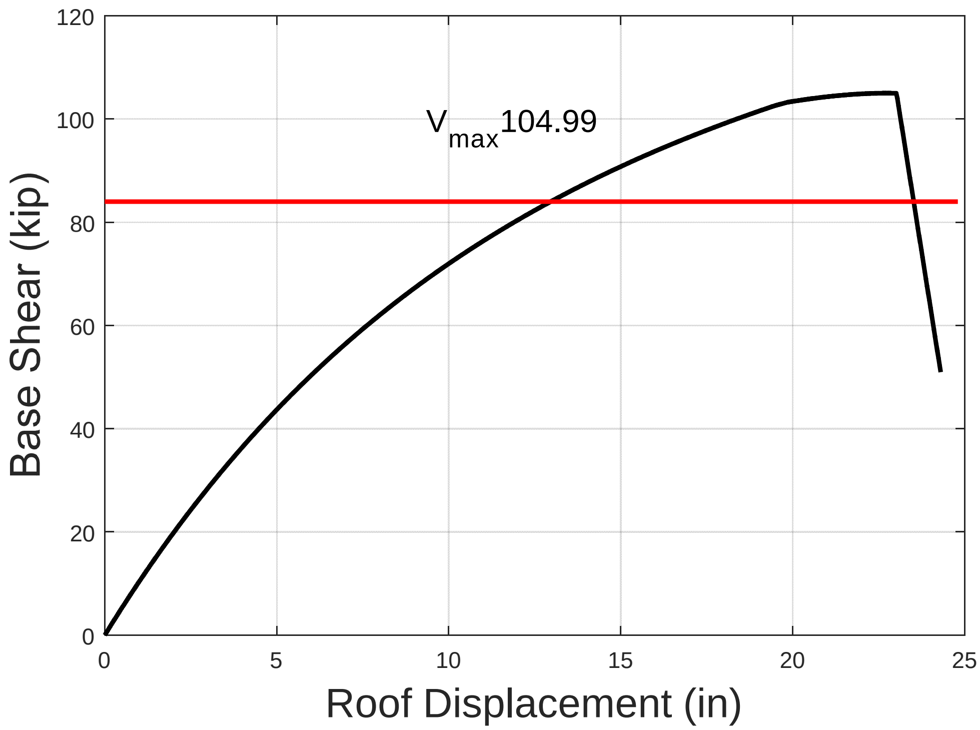


Figure G. 55. Archetype 52, R=3, static pushover

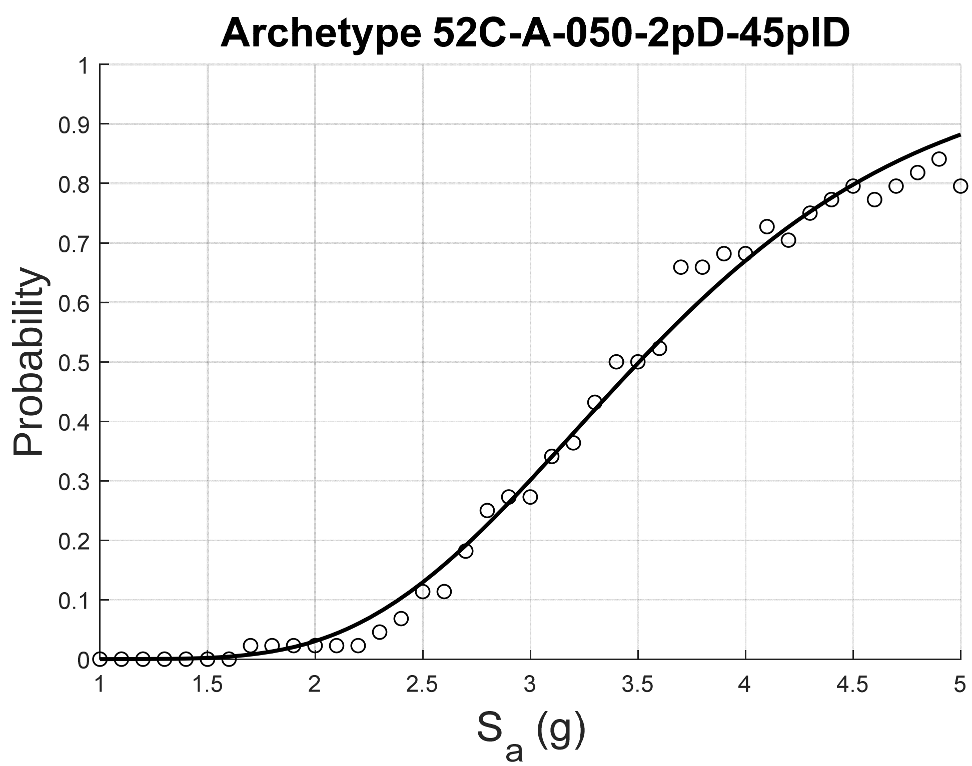


Figure G. 56. Archetype 52 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

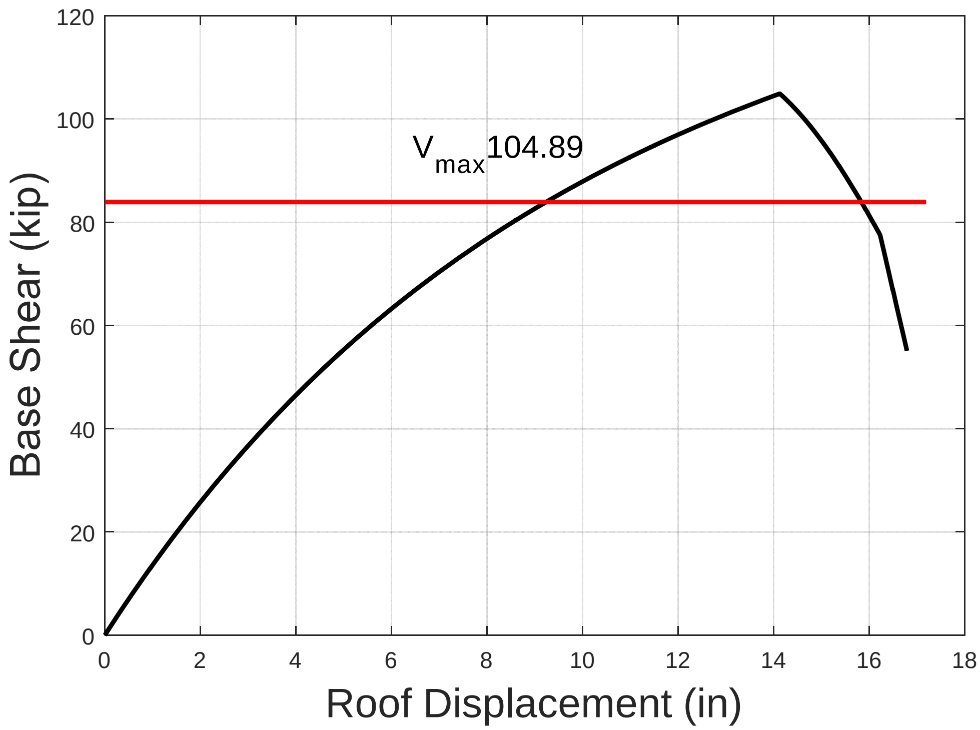


Figure G. 57. Archetype 53, R=3, static pushover

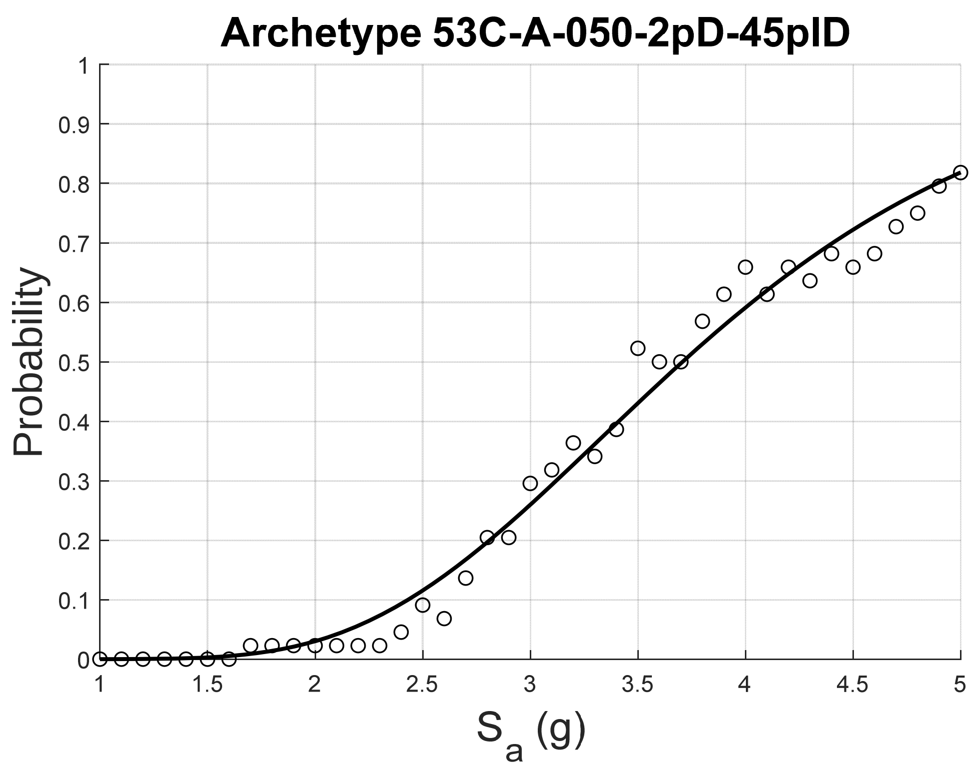


Figure G. 58. Archetype 53 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

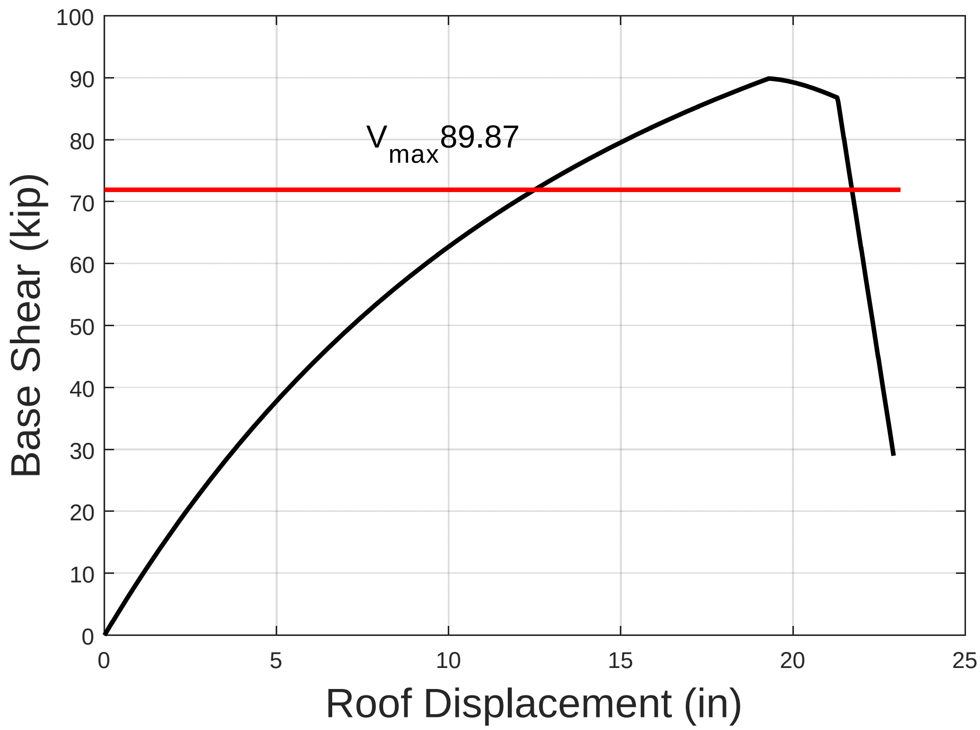


Figure G. 59. Archetype 54, R=3, static pushover

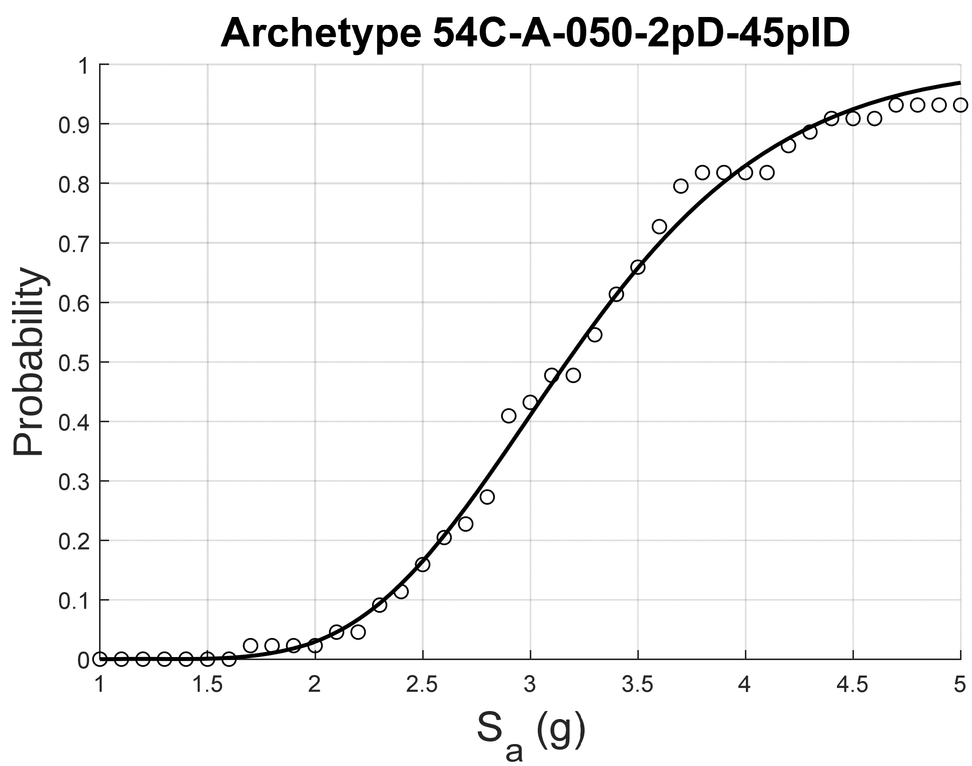


Figure G. 60. Archetype 54 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

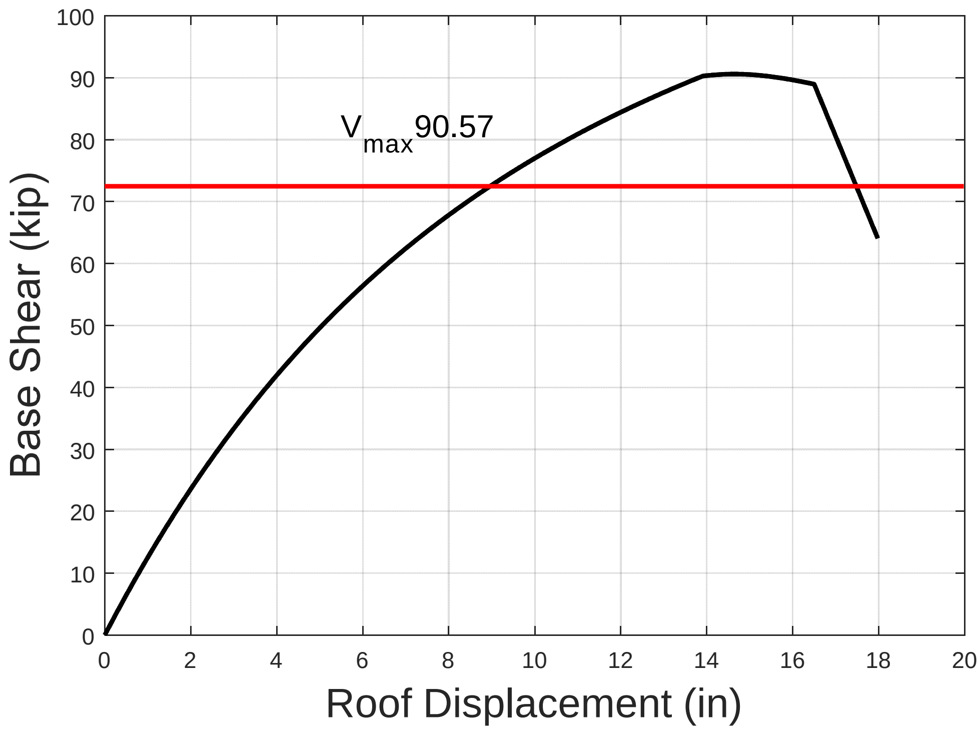


Figure G. 61. Archetype 61, R=3, static pushover

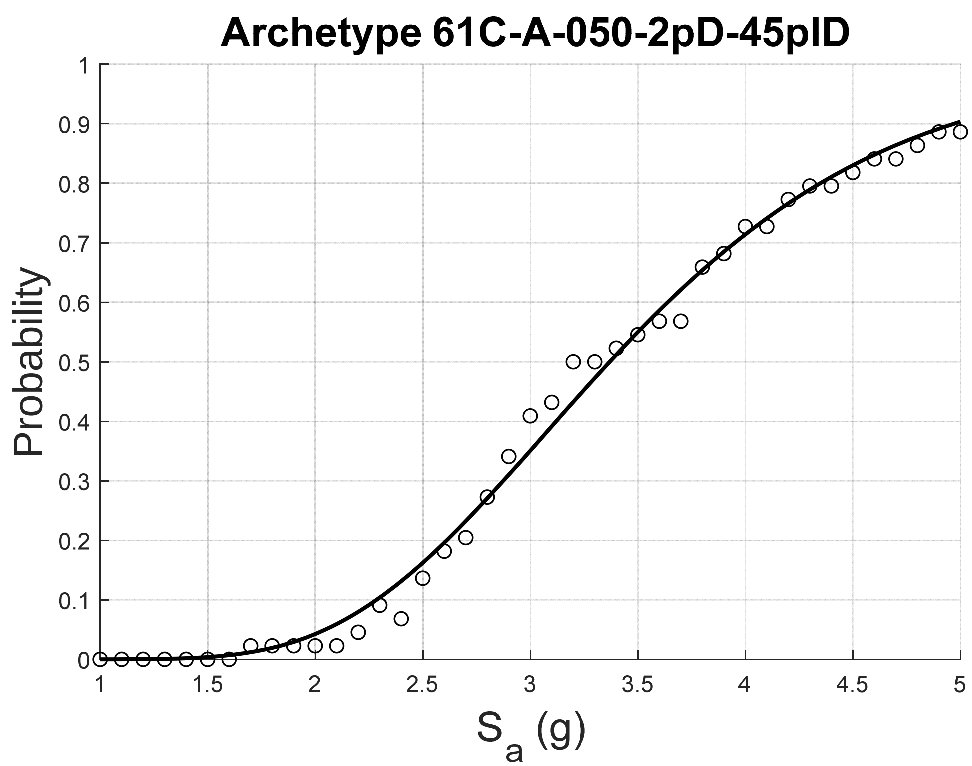


Figure G. 62. Archetype 61 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

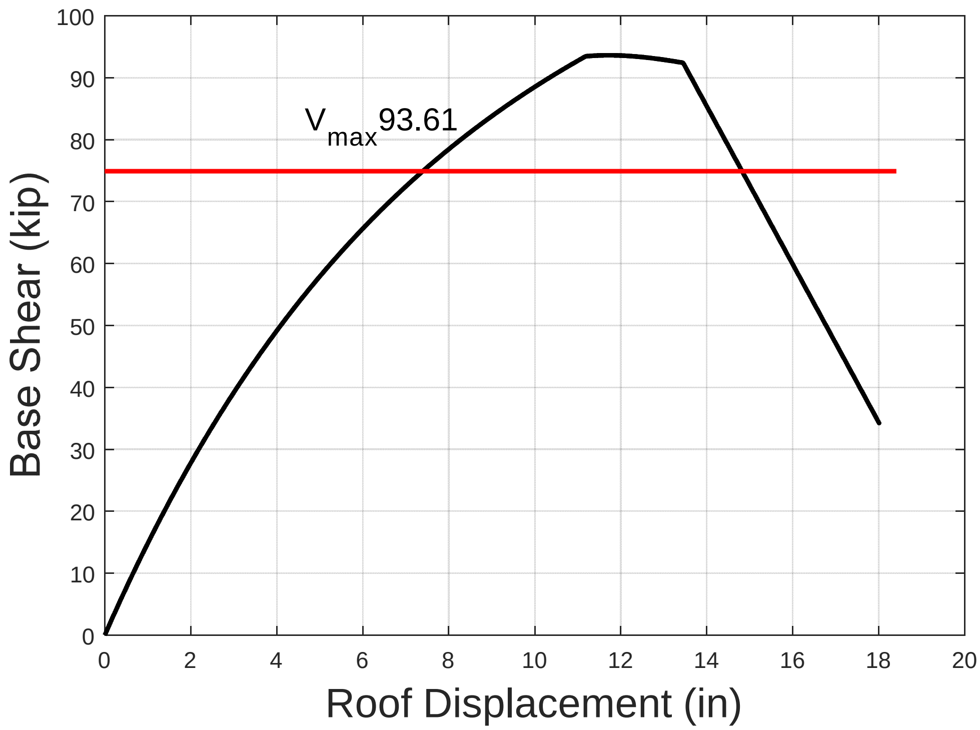


Figure G. 63. Archetype 62, R=3, static pushover

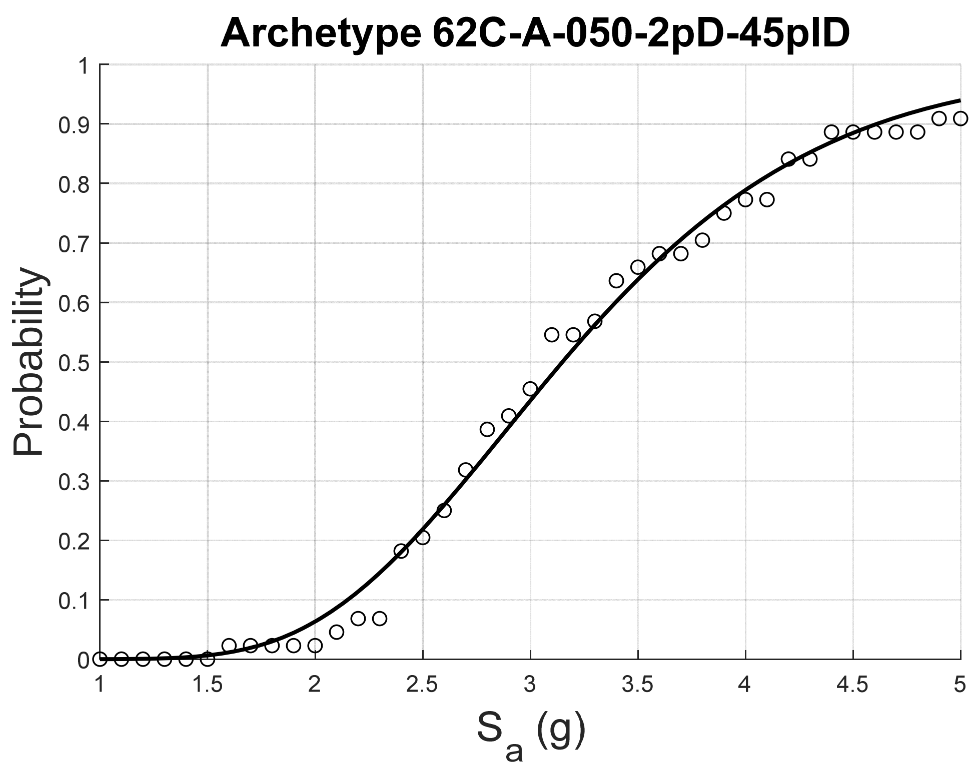


Figure G. 64. Archetype 62 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

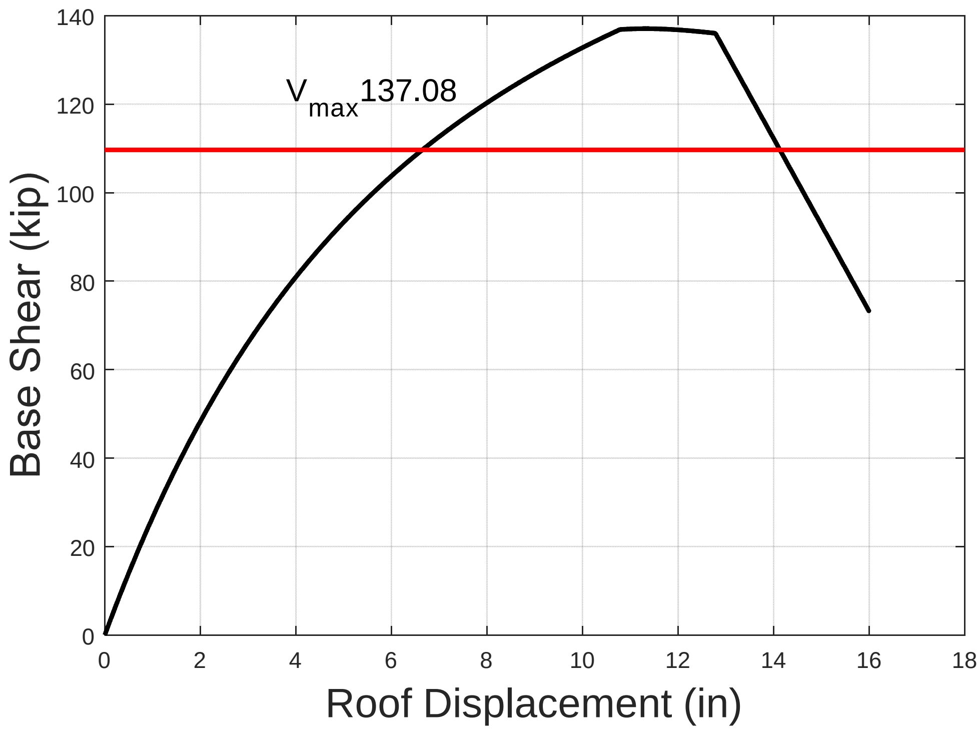


Figure G. 65. Archetype 63, R=3, static pushover

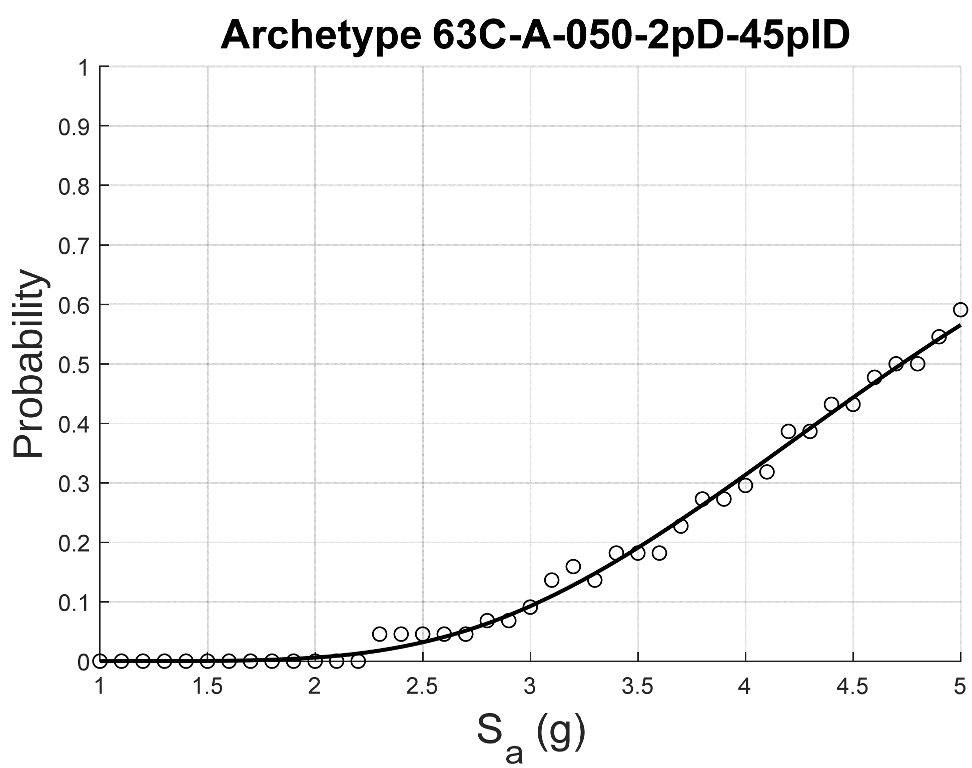


Figure G. 66. Archetype 63 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

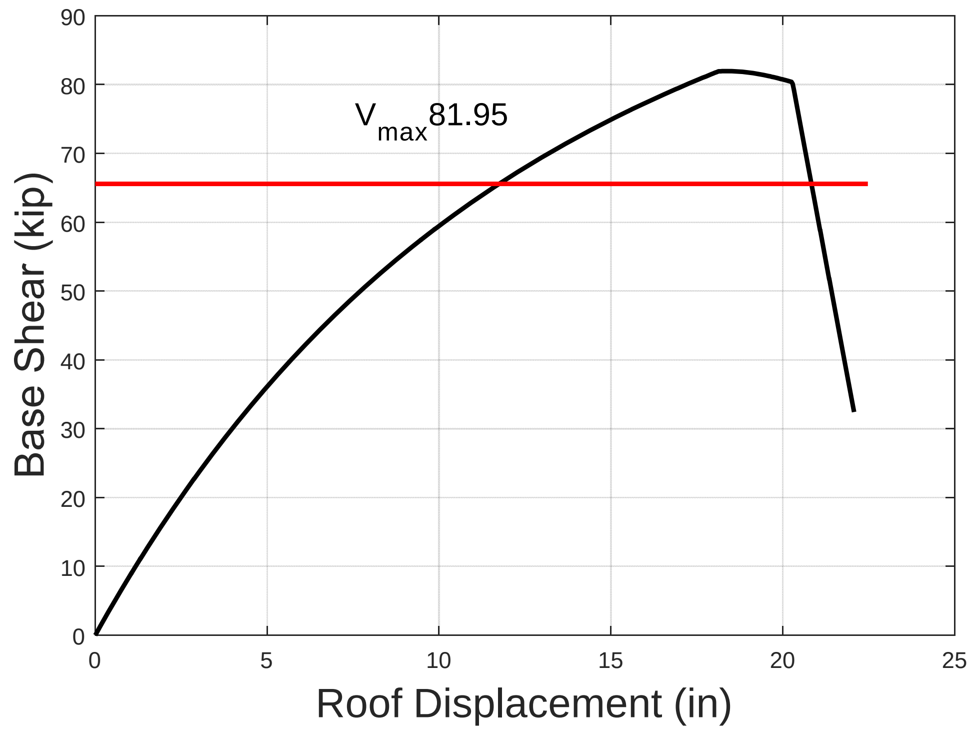


Figure G. 67. Archetype 64, R=3, static pushover

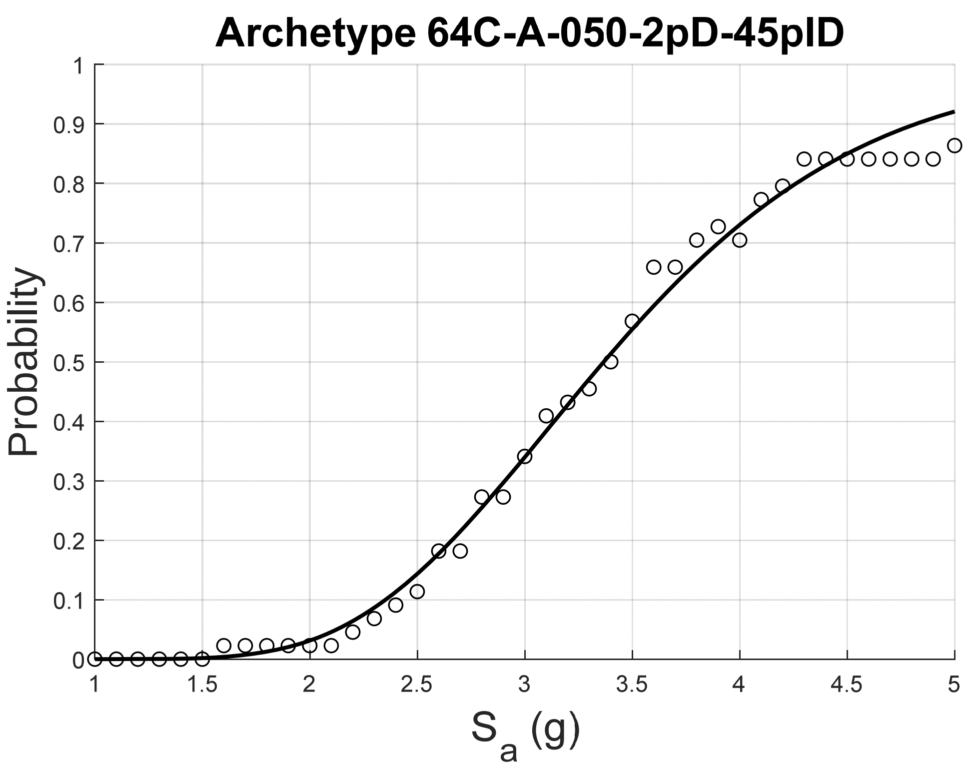


Figure G. 68. Archetype 64 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

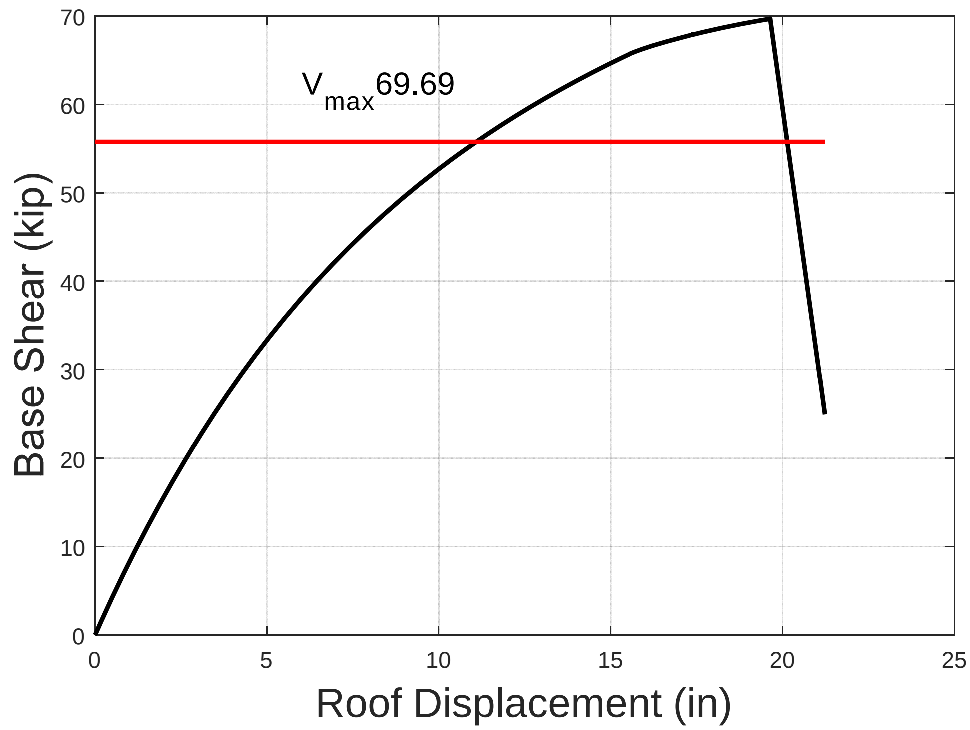


Figure G. 69. Archetype 65, R=3, static pushover

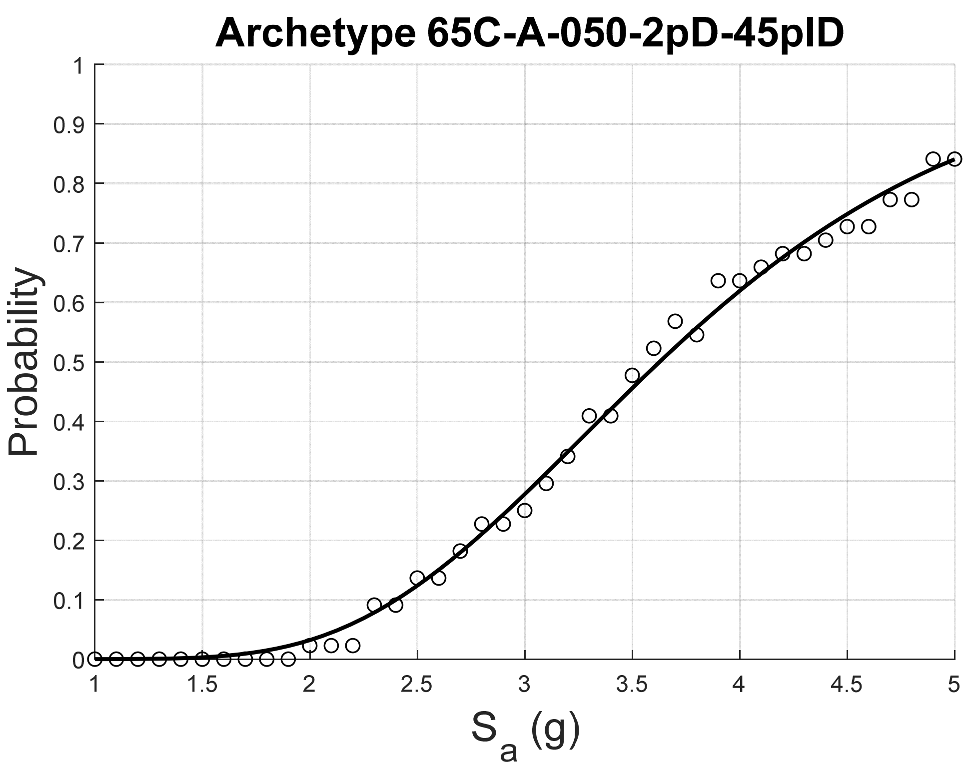


Figure G. 70. Archetype 65 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

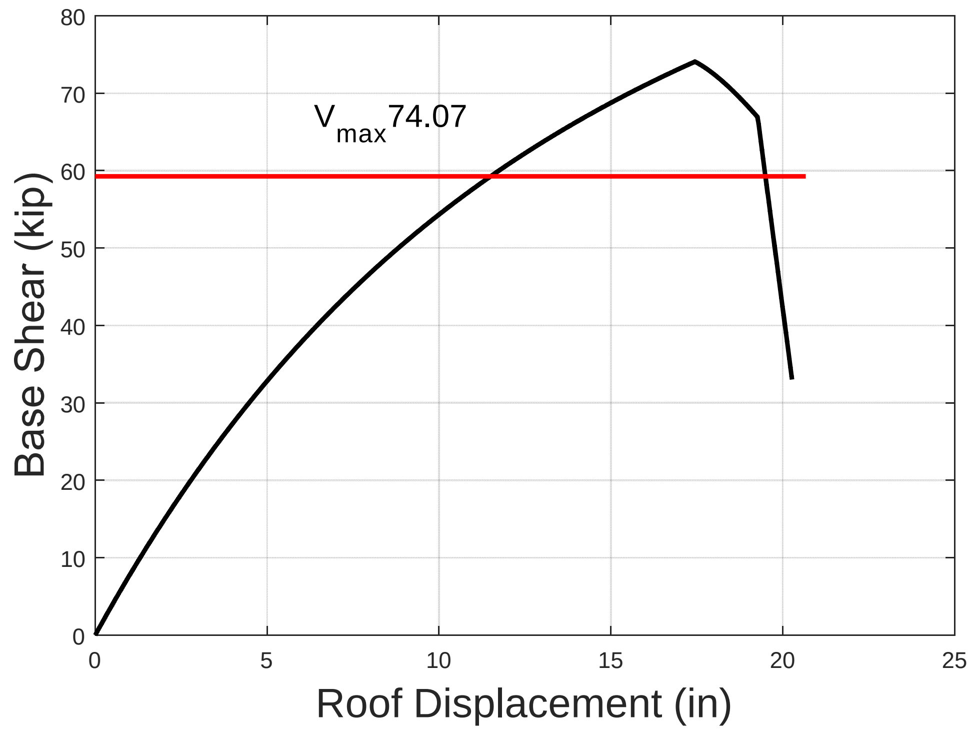


Figure G. 71. Archetype 66, R=3, static pushover

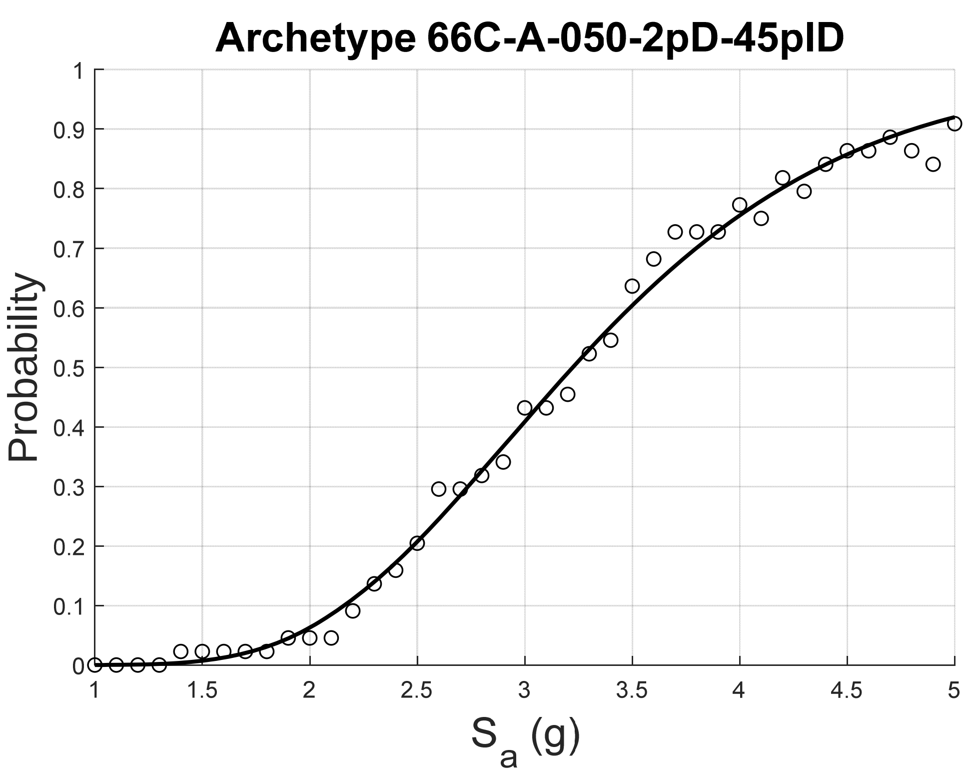


Figure G. 72. Archetype 66 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

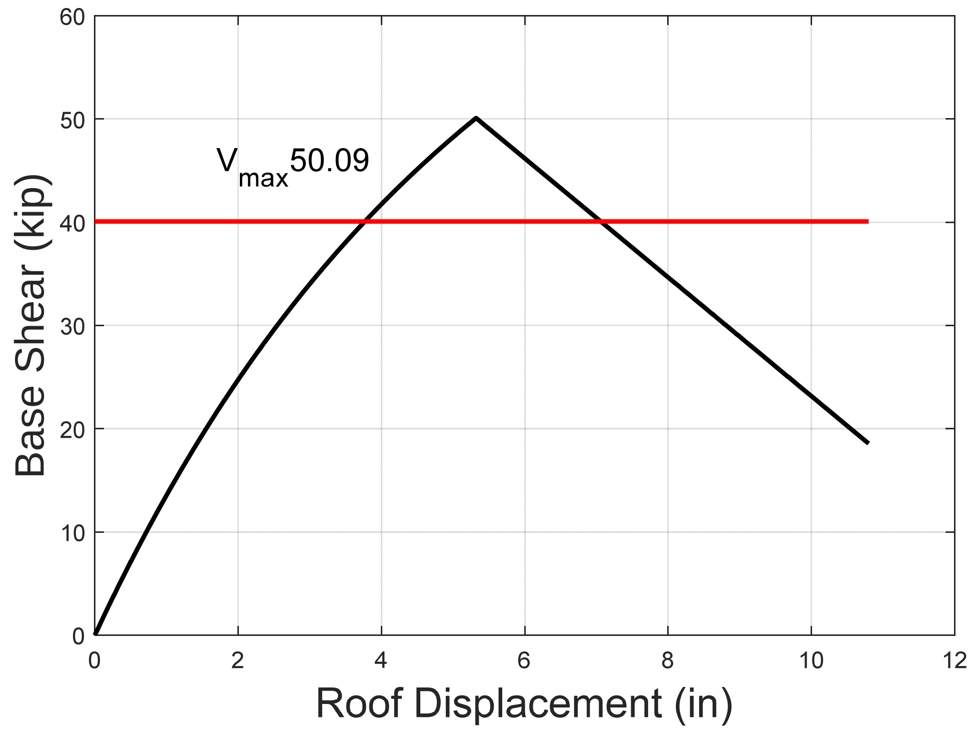


Figure G. 73. Archetype 73, R=3, static pushover

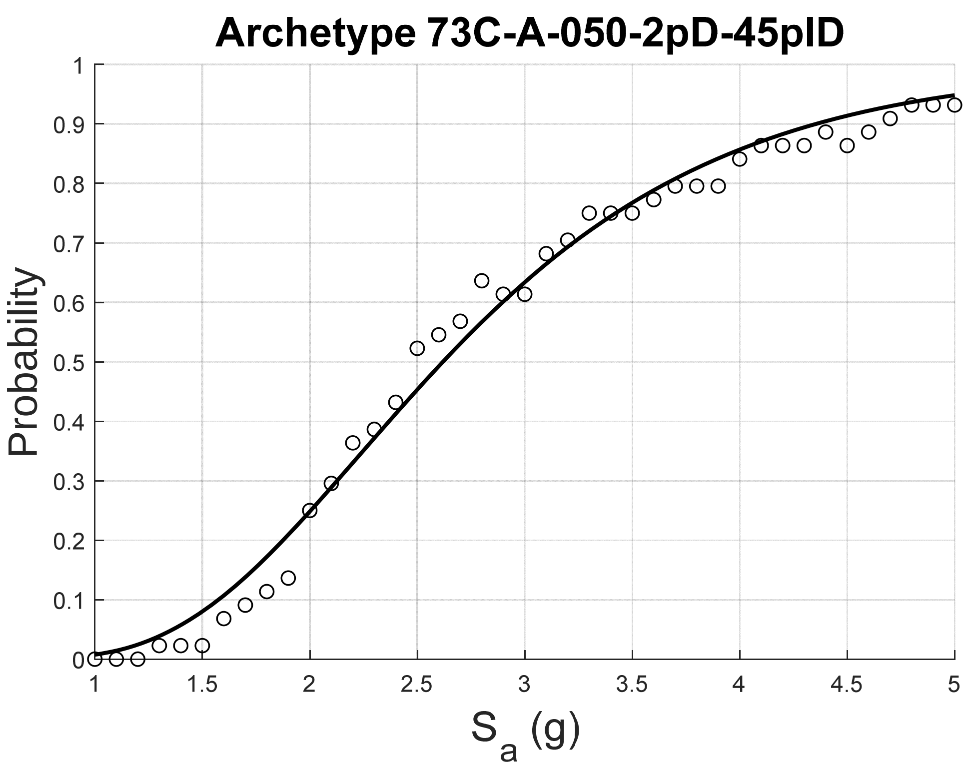


Figure G. 74. Archetype 73 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

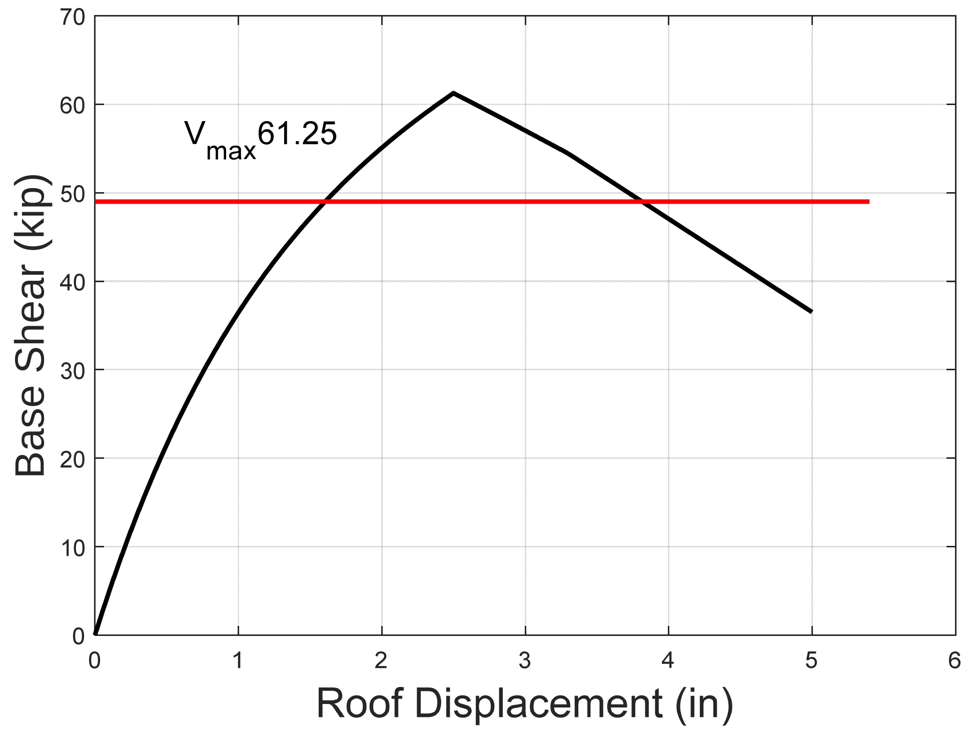


Figure G. 75. Archetype 74, R=3, static pushover

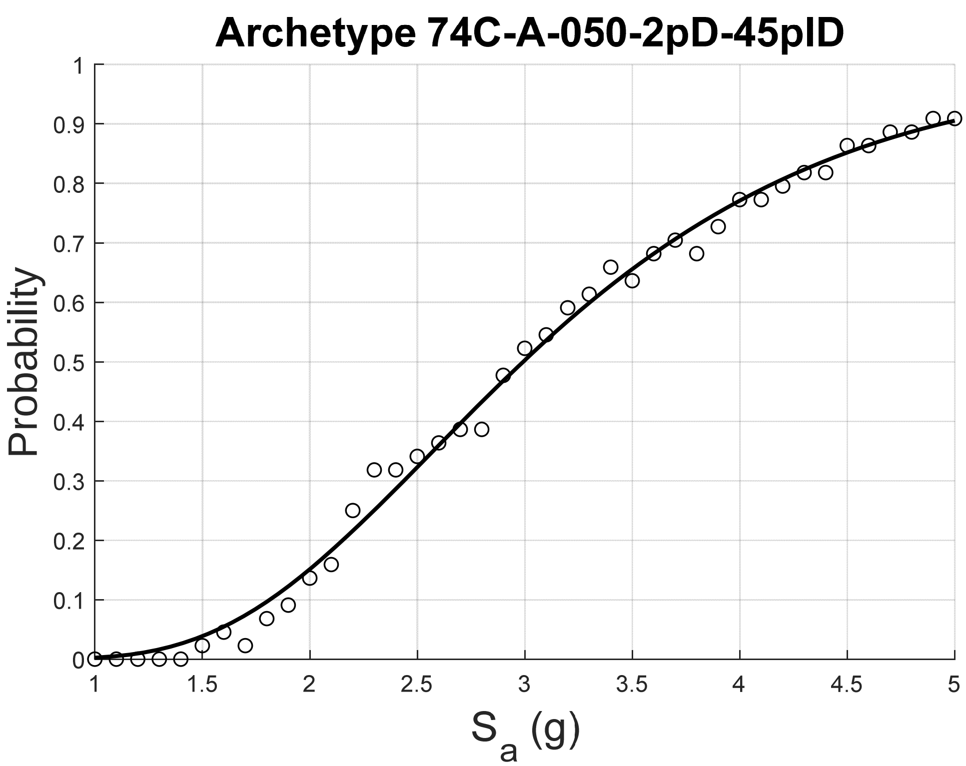


Figure G. 76. Archetype 74 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

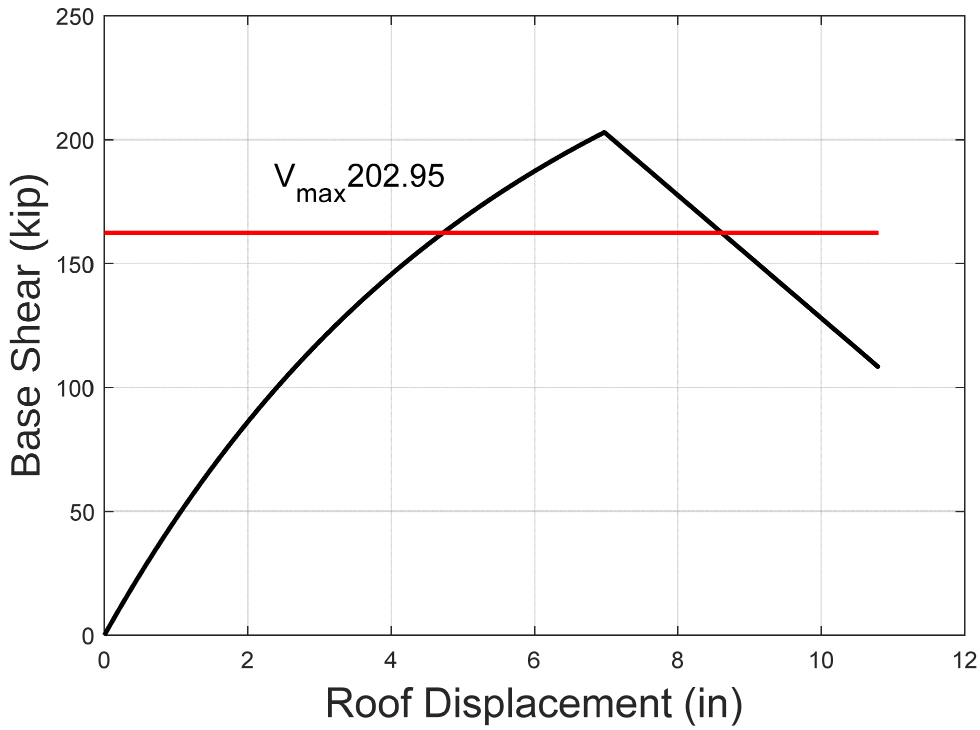


Figure G. 77. Archetype 75, R=3, static pushover

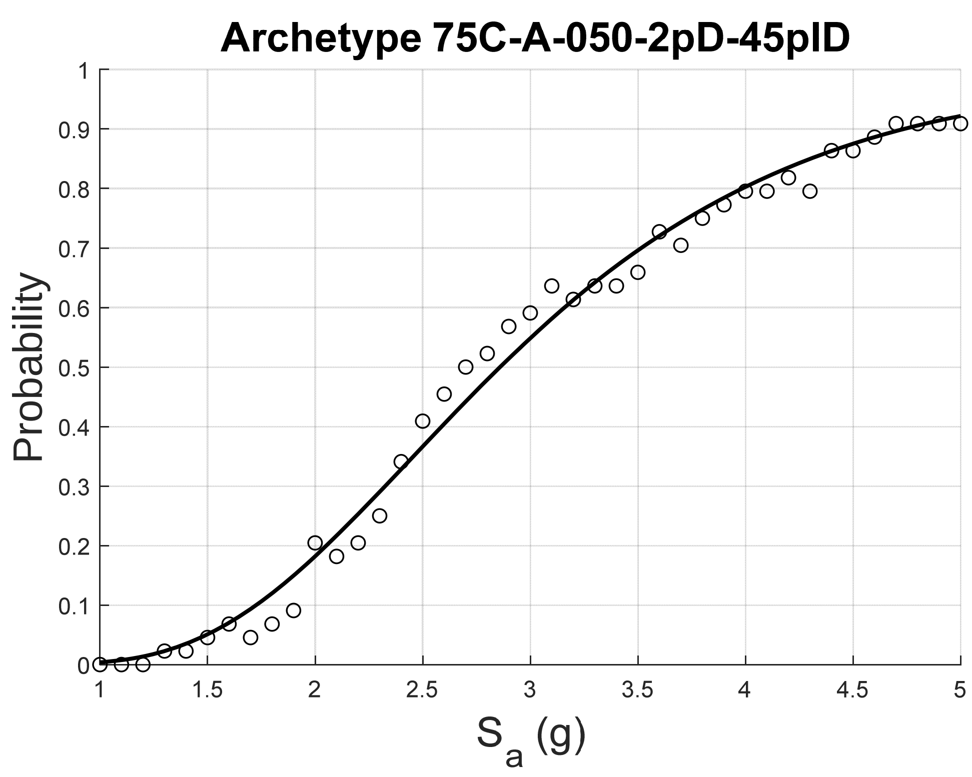


Figure G. 78. Archetype 75 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

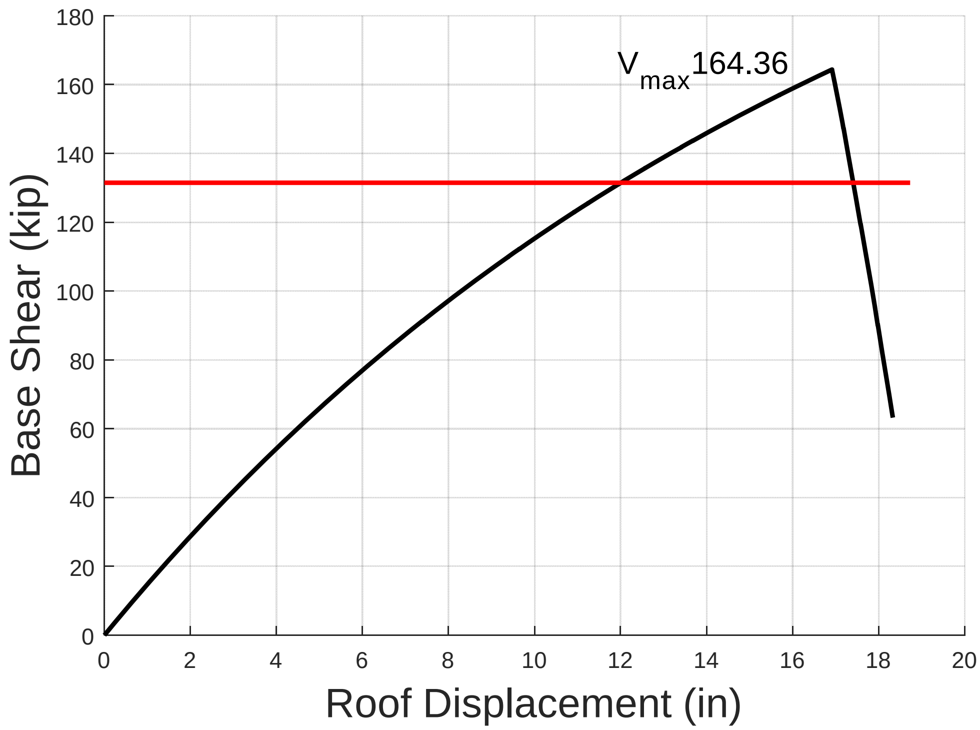


Figure G. 79. Archetype 76, R=3, static pushover

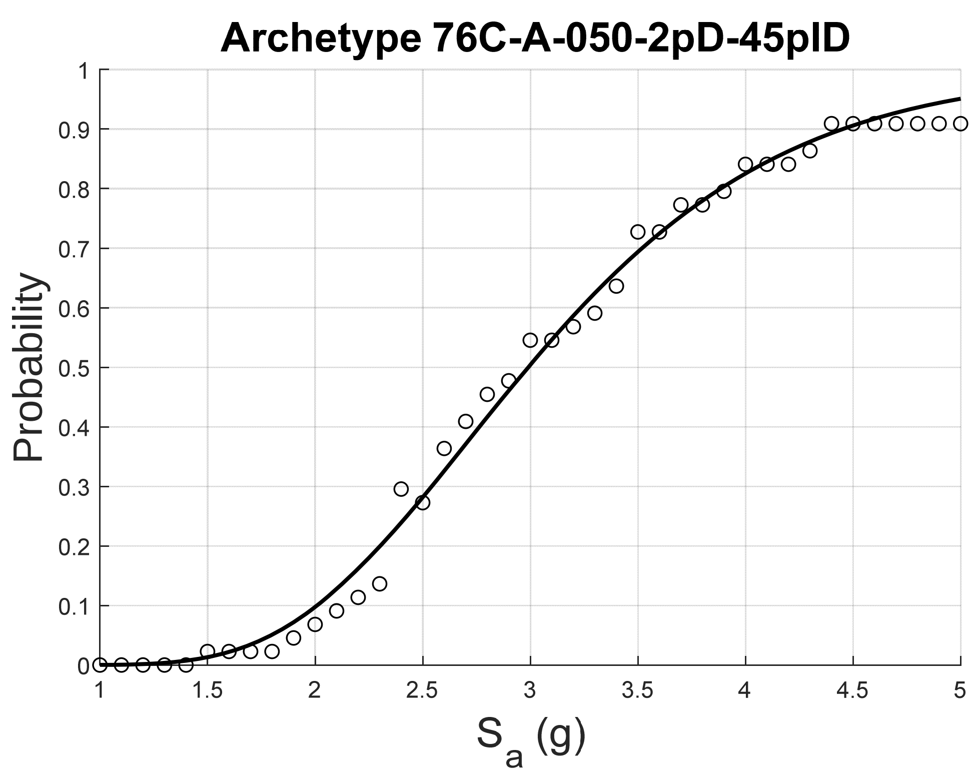


Figure G. 80. Archetype 76 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

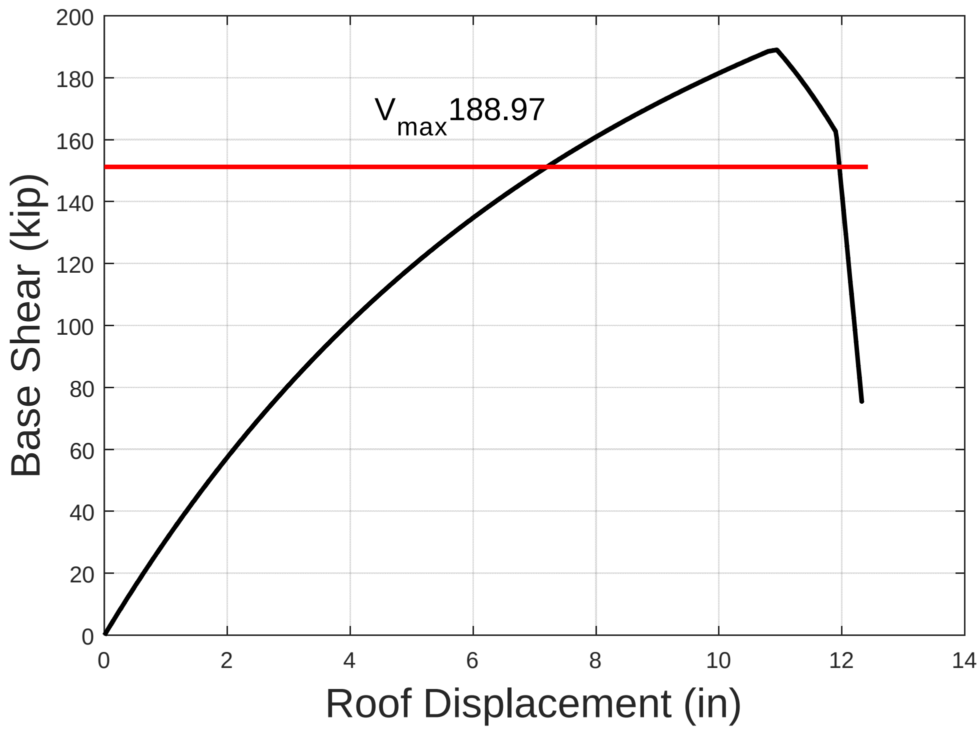


Figure G. 81. Archetype 77, R=3, static pushover

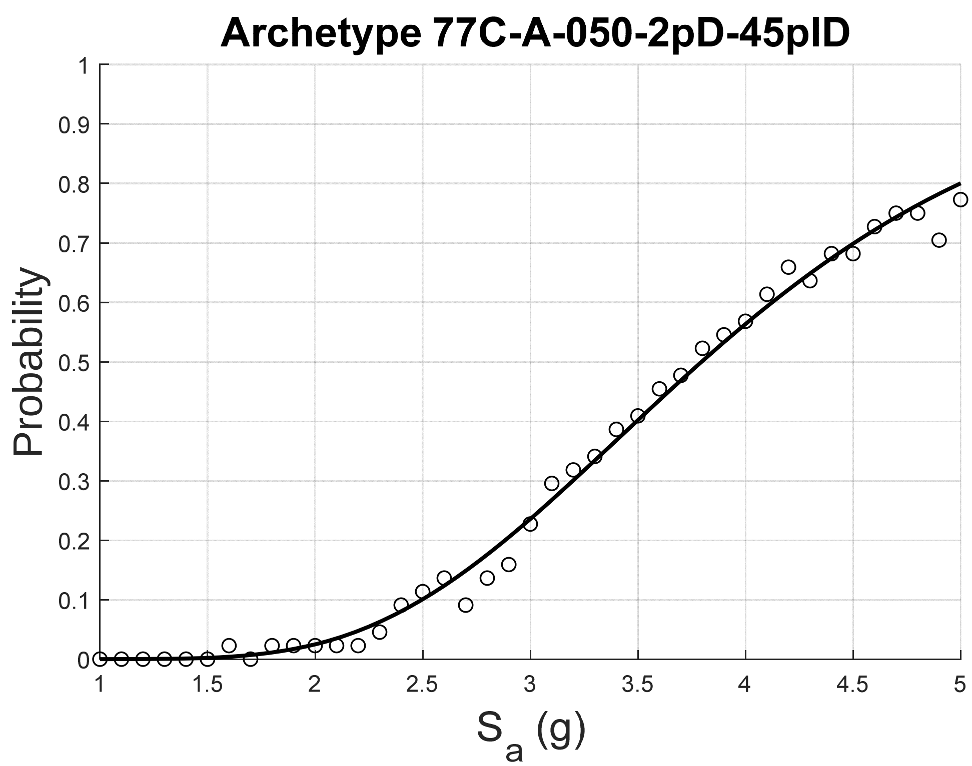


Figure G. 82. Archetype 77 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

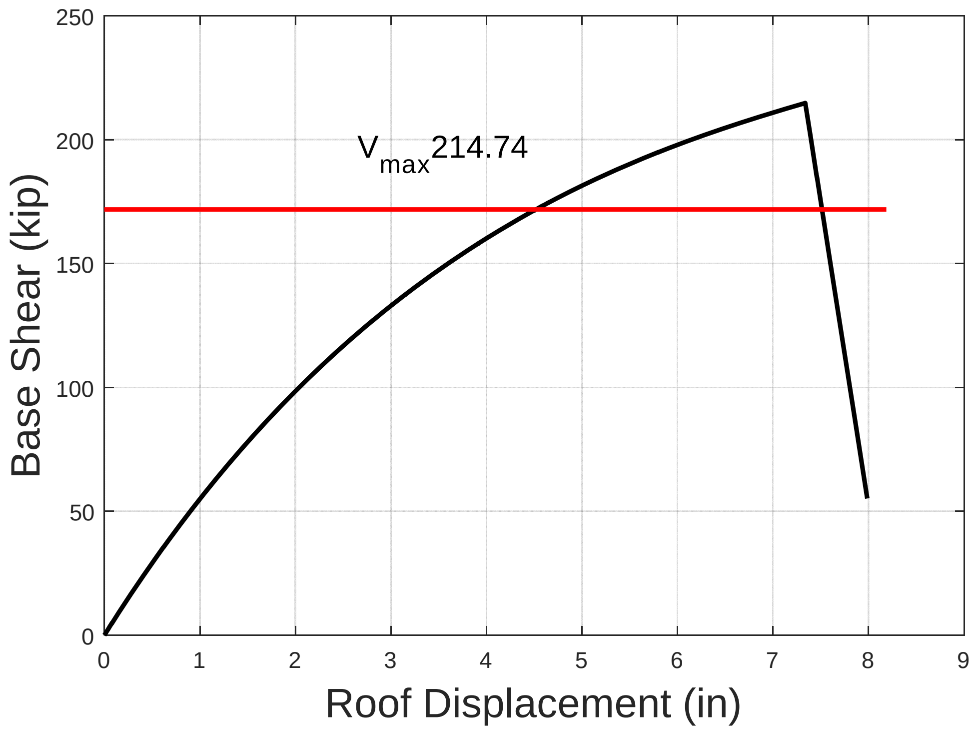


Figure G. 83. Archetype 78, R=3, static pushover

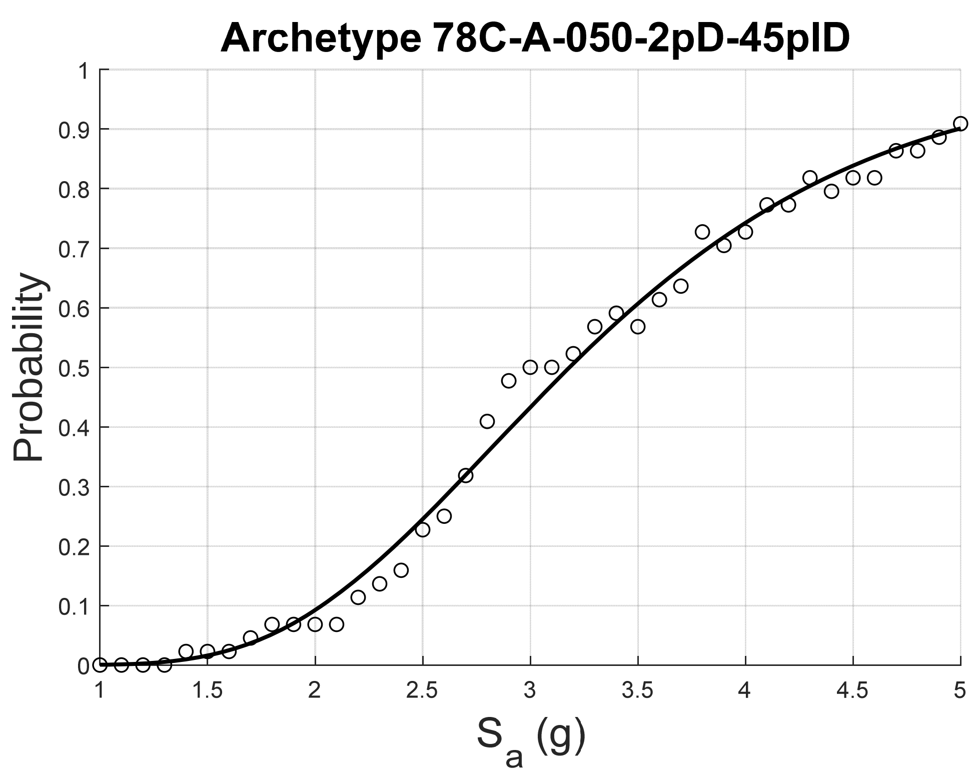


Figure G. 84. Archetype 78 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

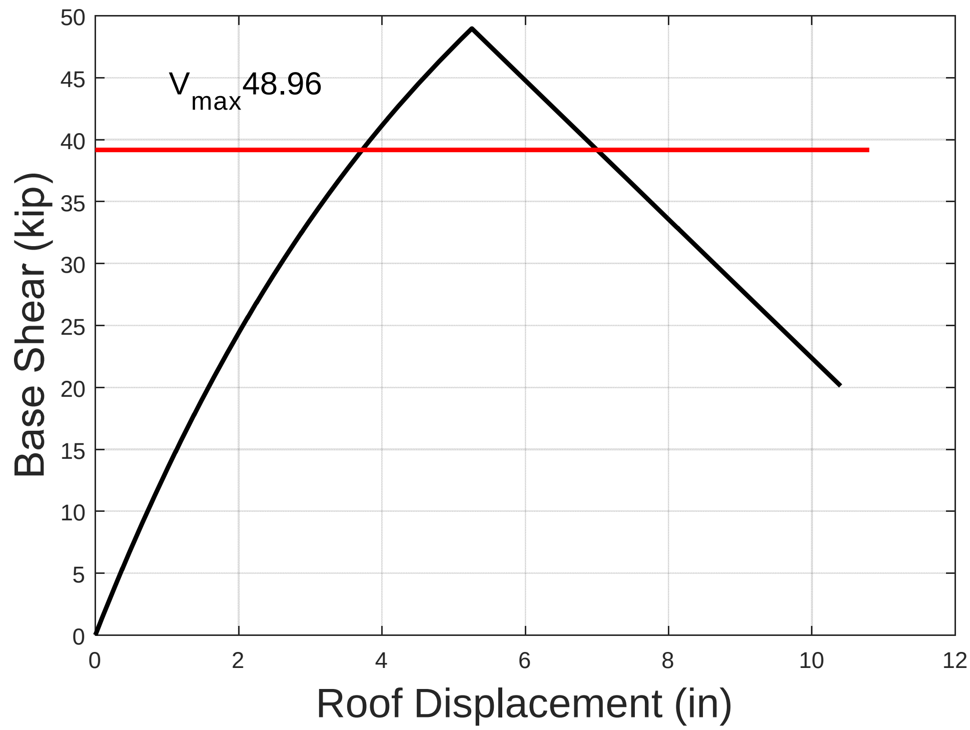


Figure G. 85. Archetype 85, R=3, static pushover

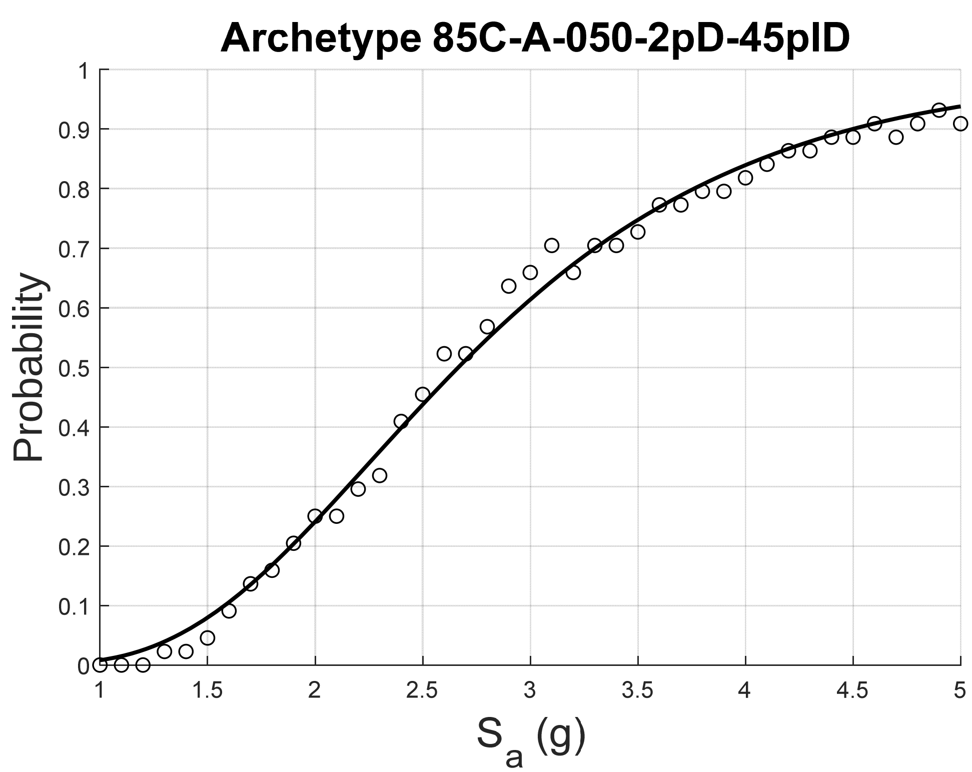


Figure G. 86. Archetype 85 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

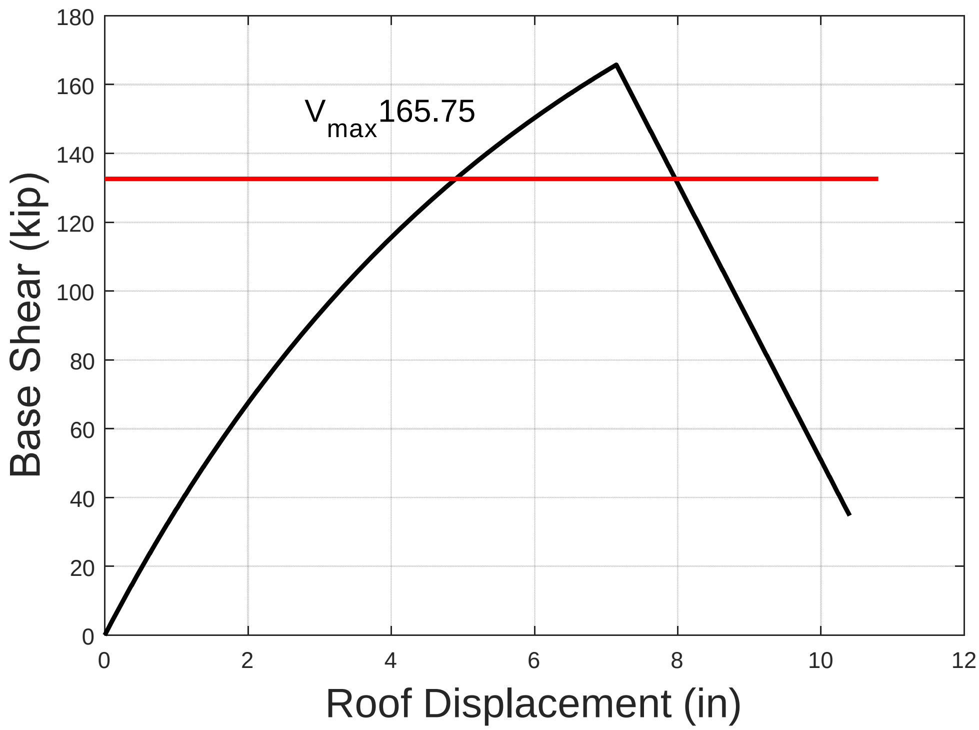


Figure G. 87. Archetype 87, R=3, static pushover

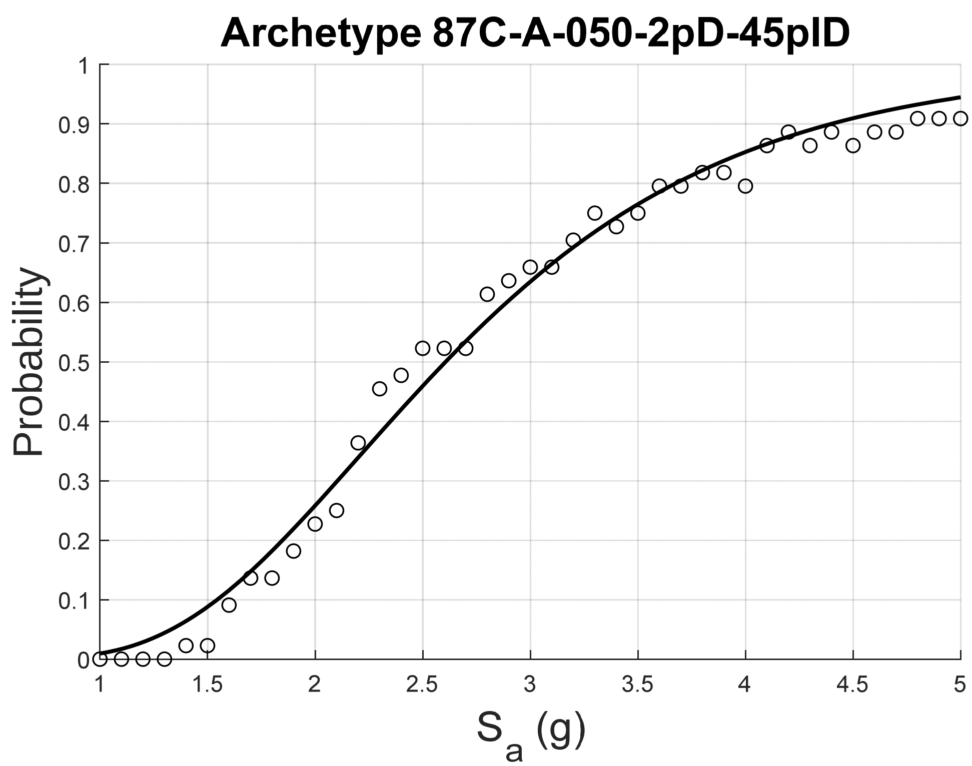


Figure G. 88. Archetype 87 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

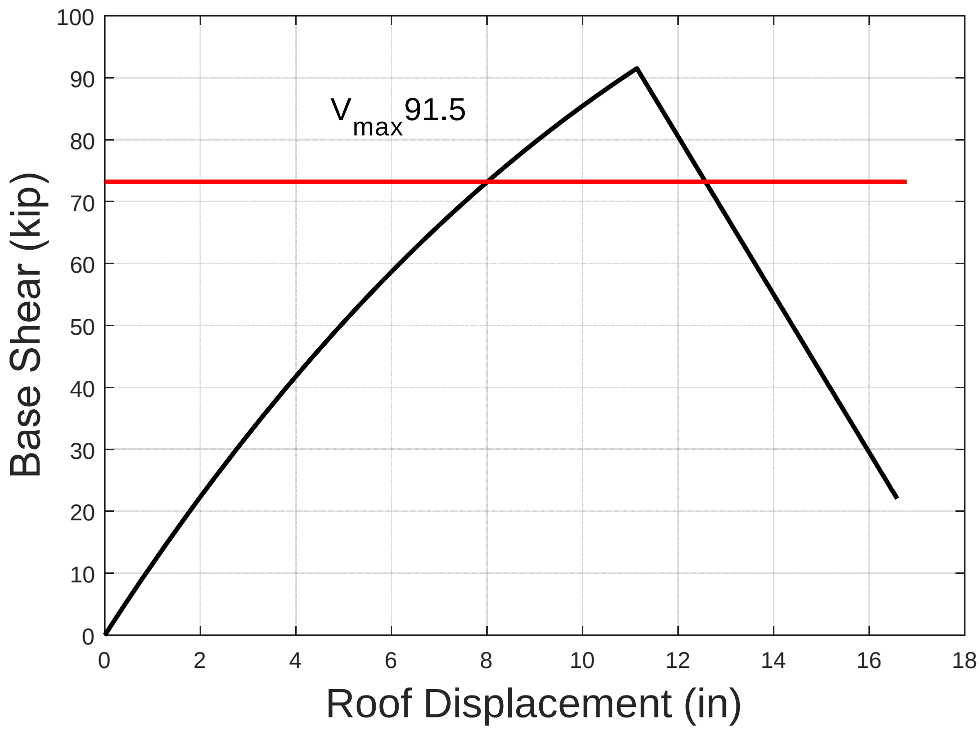


Figure G. 89. Archetype 88, R=3, static pushover

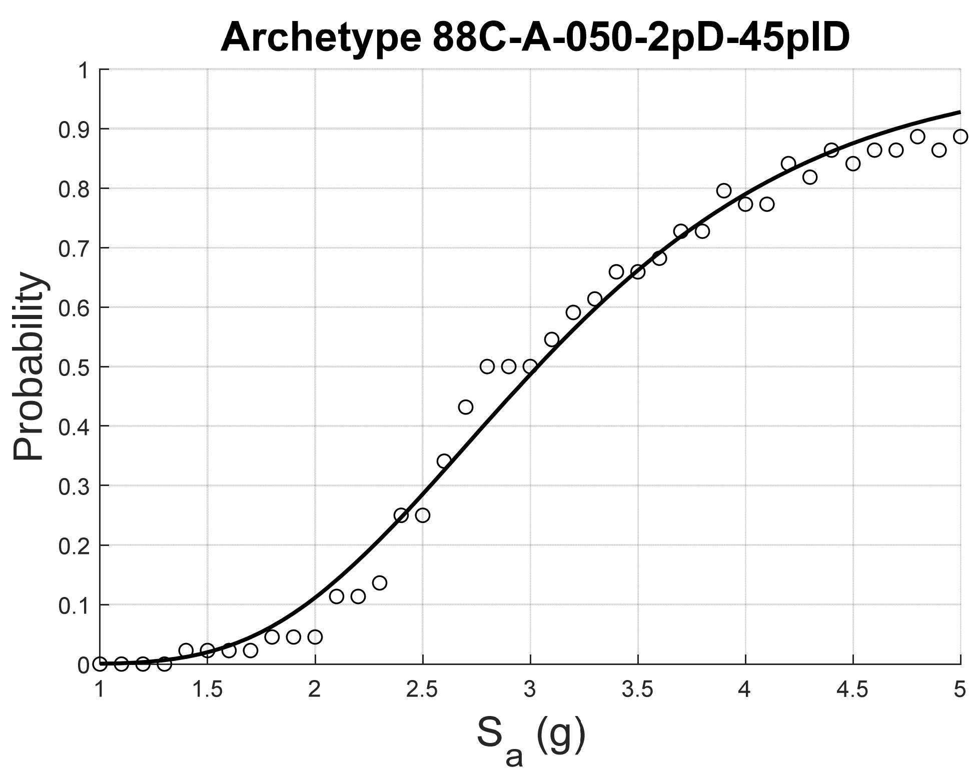


Figure G. 90. Archetype 88 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

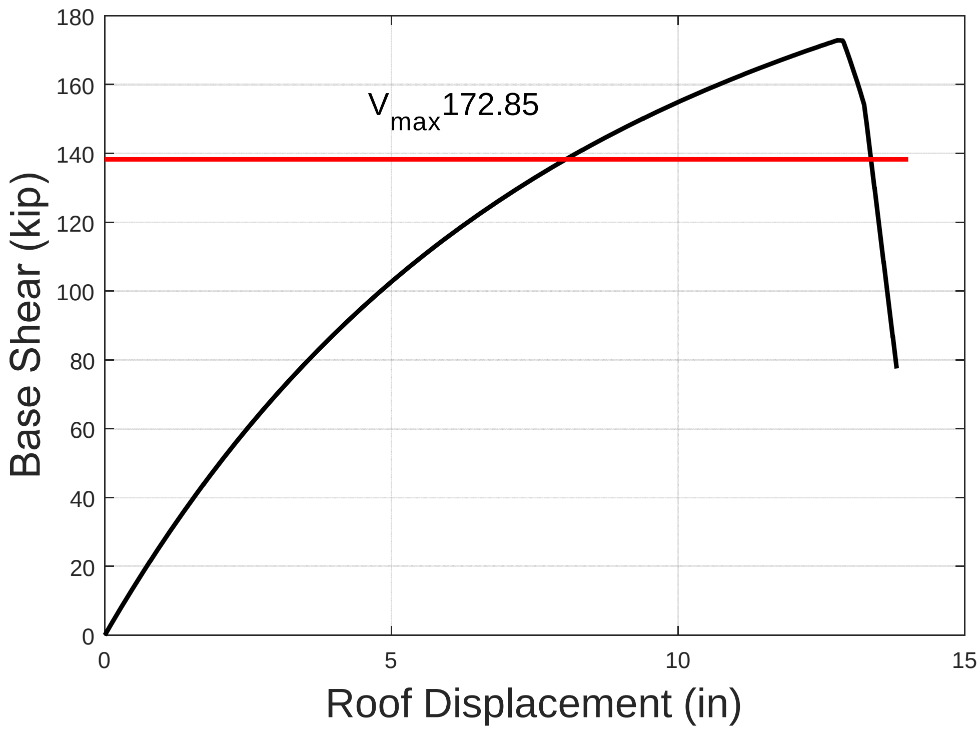


Figure G. 91. Archetype 89, R=3, static pushover

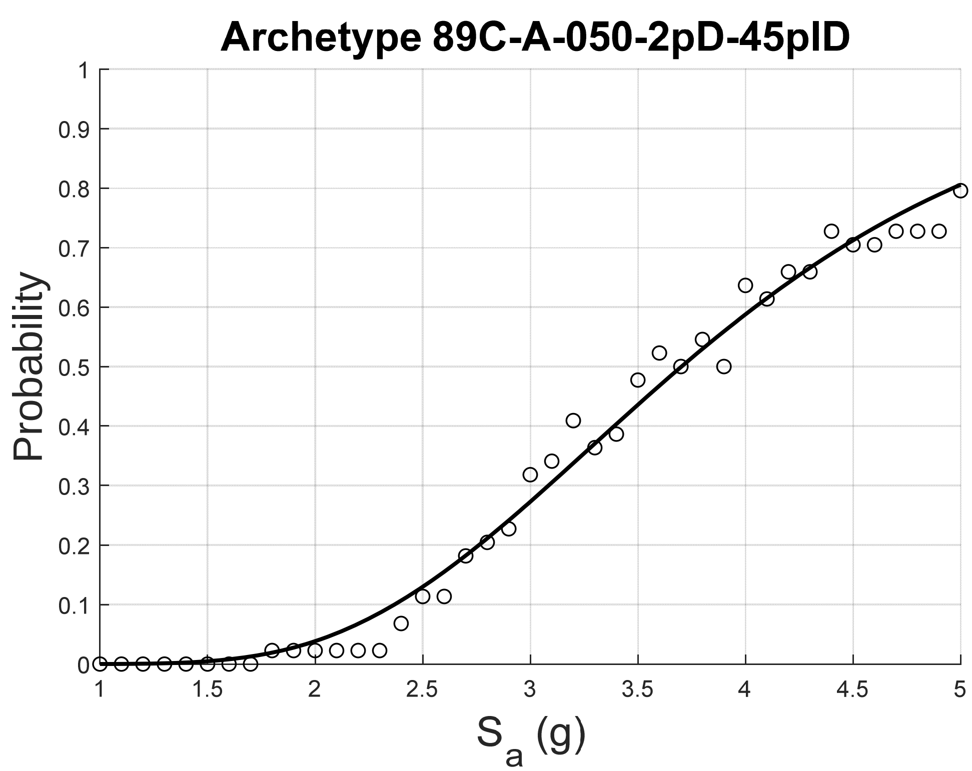


Figure G. 92. Archetype 89 collapse fragility, R=3, 2% damping, 4% interstory drift limit

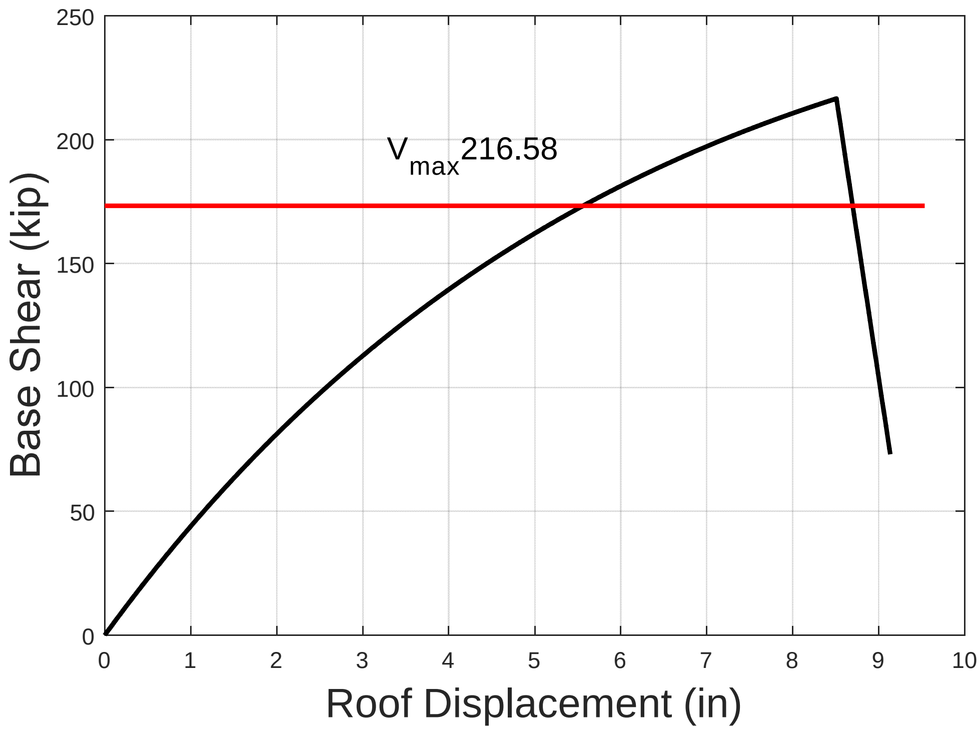


Figure G. 93. Archetype 90, R=3, static pushover

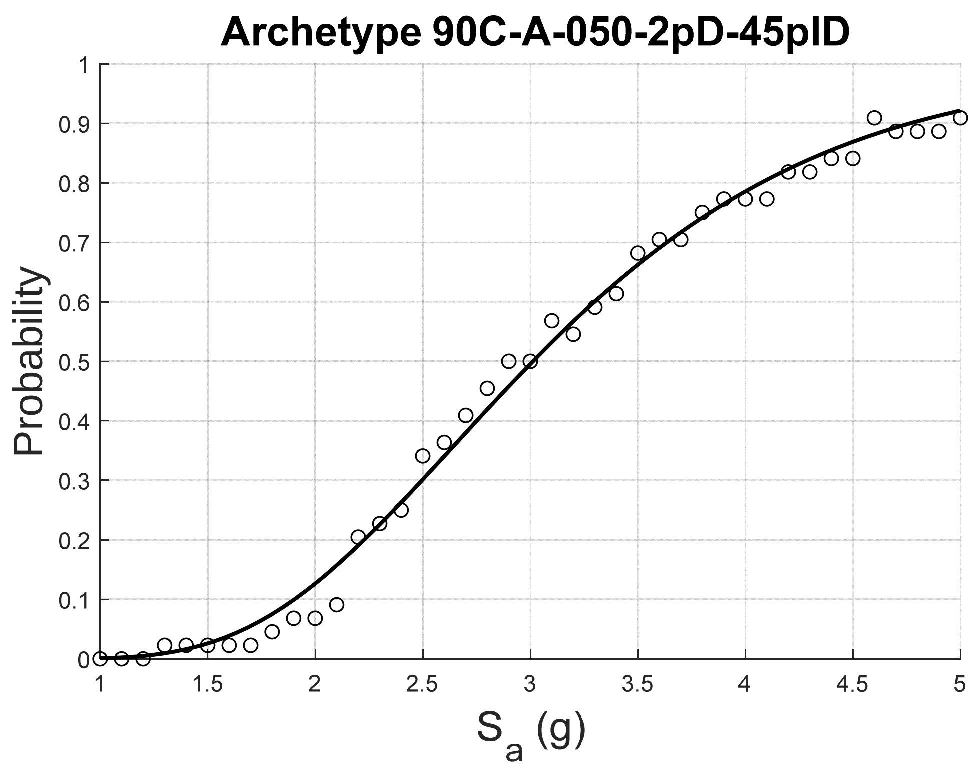


Figure G. 94. Archetype 90 collapse fragility, R=3, 2% damping, 4.5% interstory drift limit

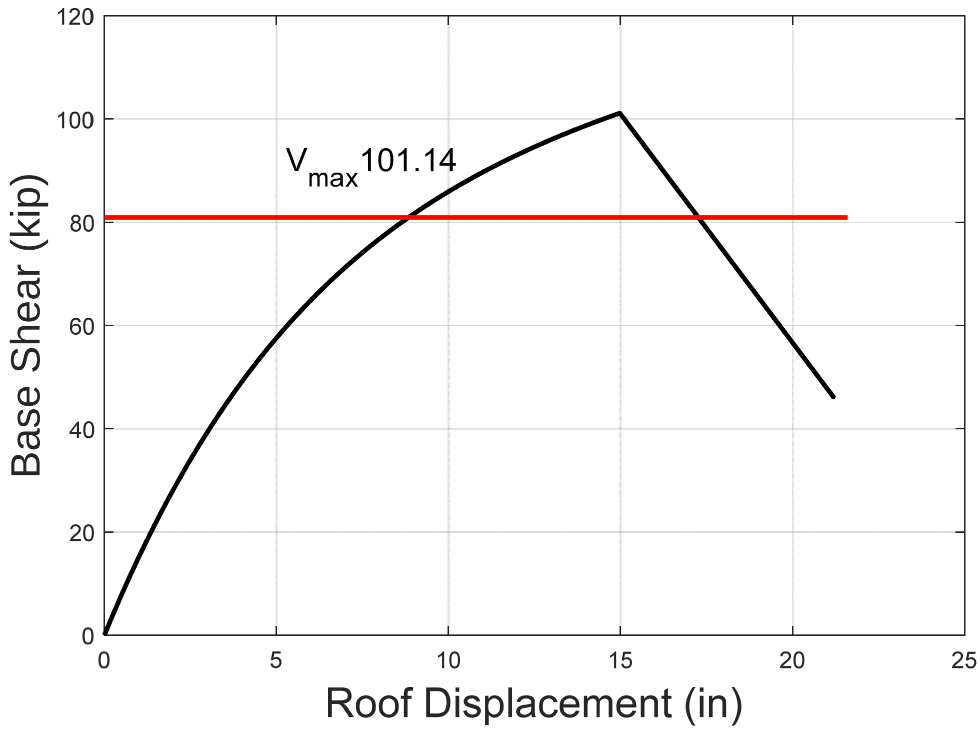


Figure G. 95. Archetype 97, R=3, static pushover

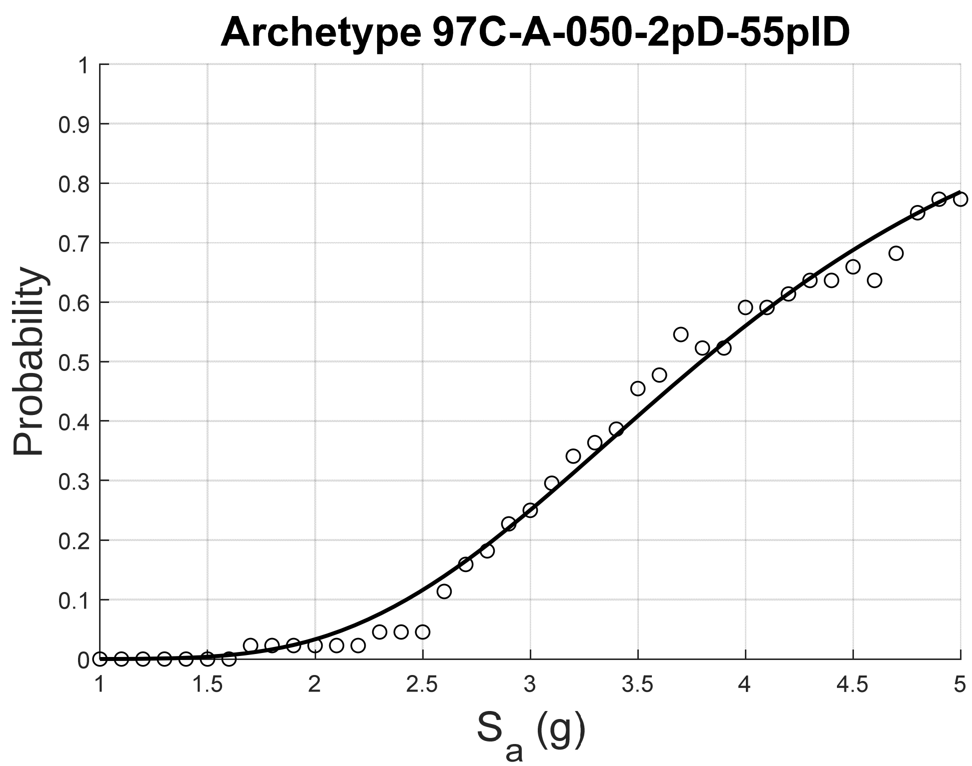


Figure G. 96. Archetype 97 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

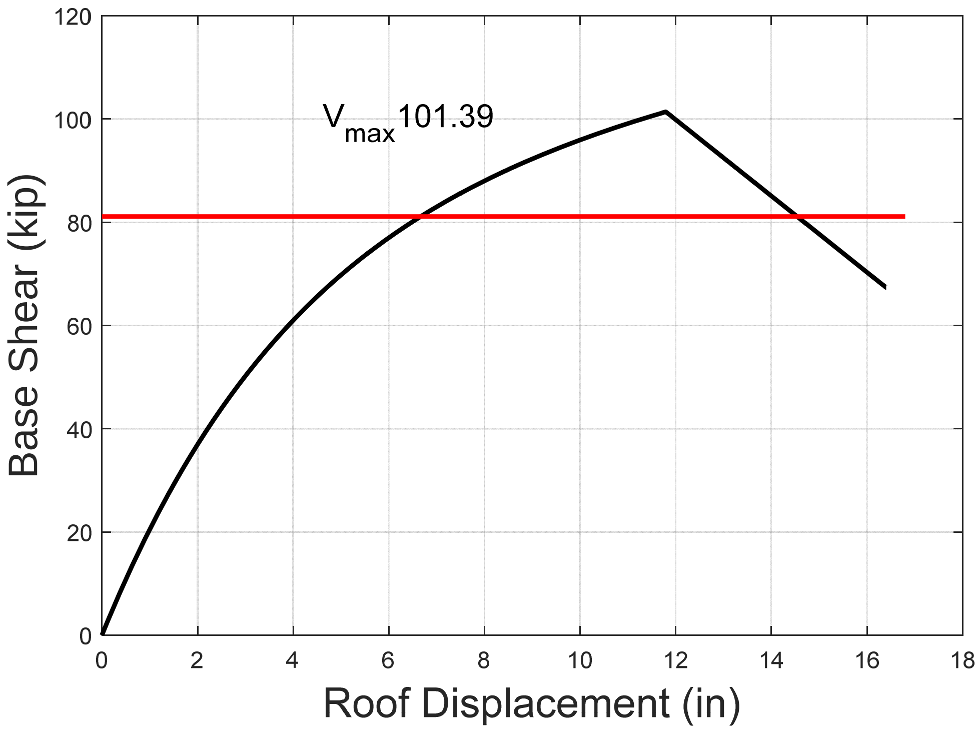


Figure G. 97. Archetype 98, R=3, static pushover

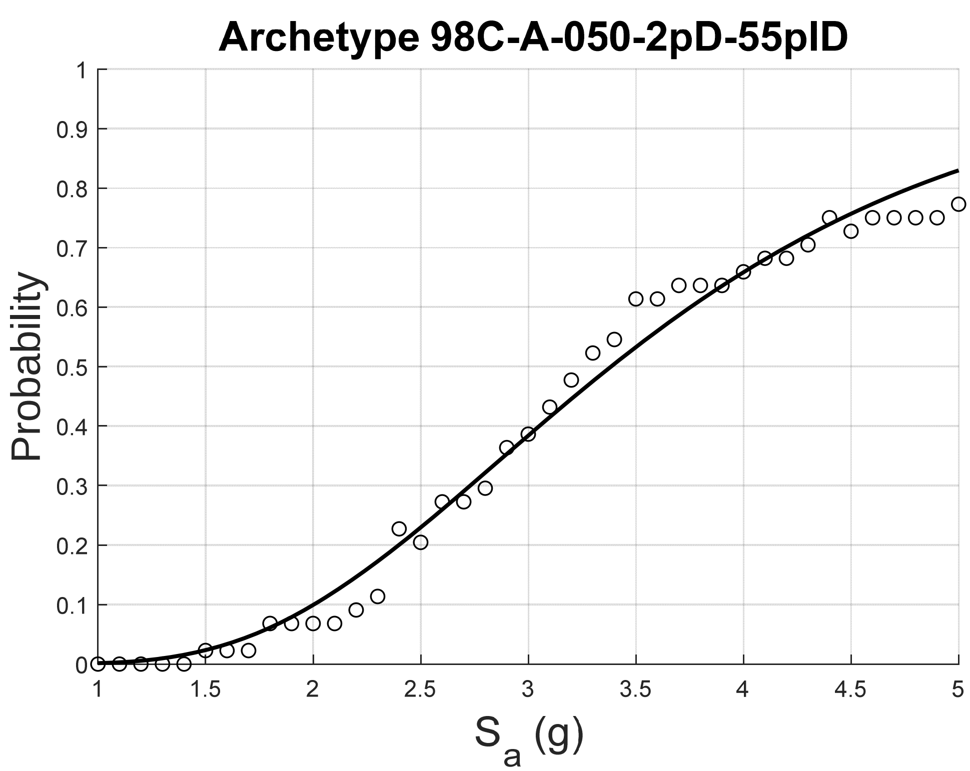


Figure G. 98. Archetype 98 collapse fragility, R=3, 2% damping, 5.5% interstory drift limit

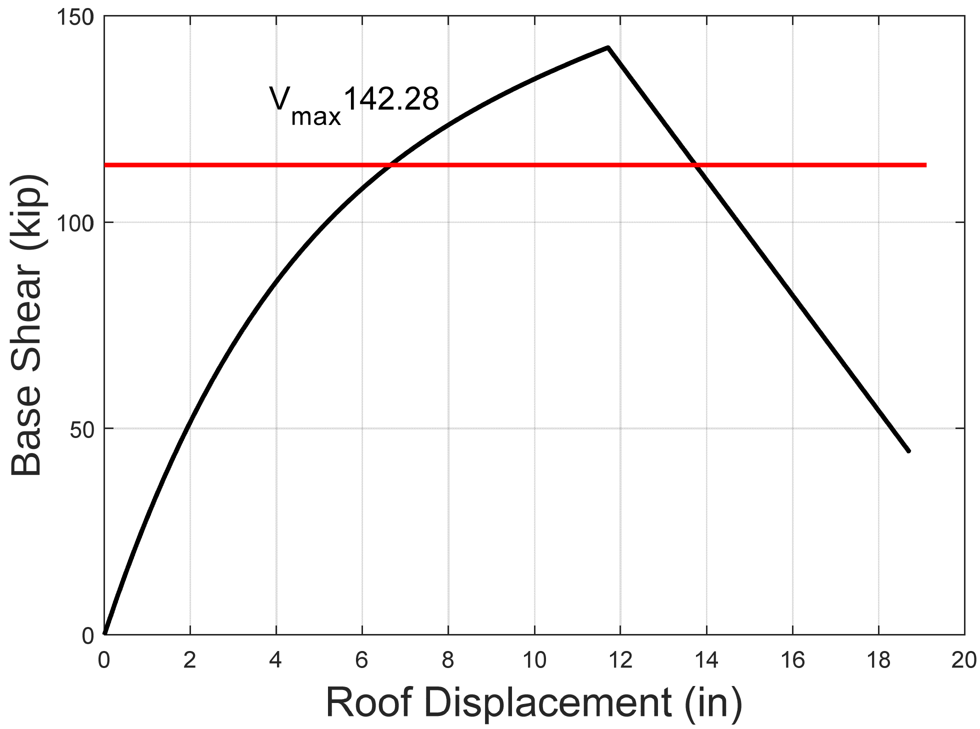


Figure G. 99. Archetype 99, R=3, static pushover

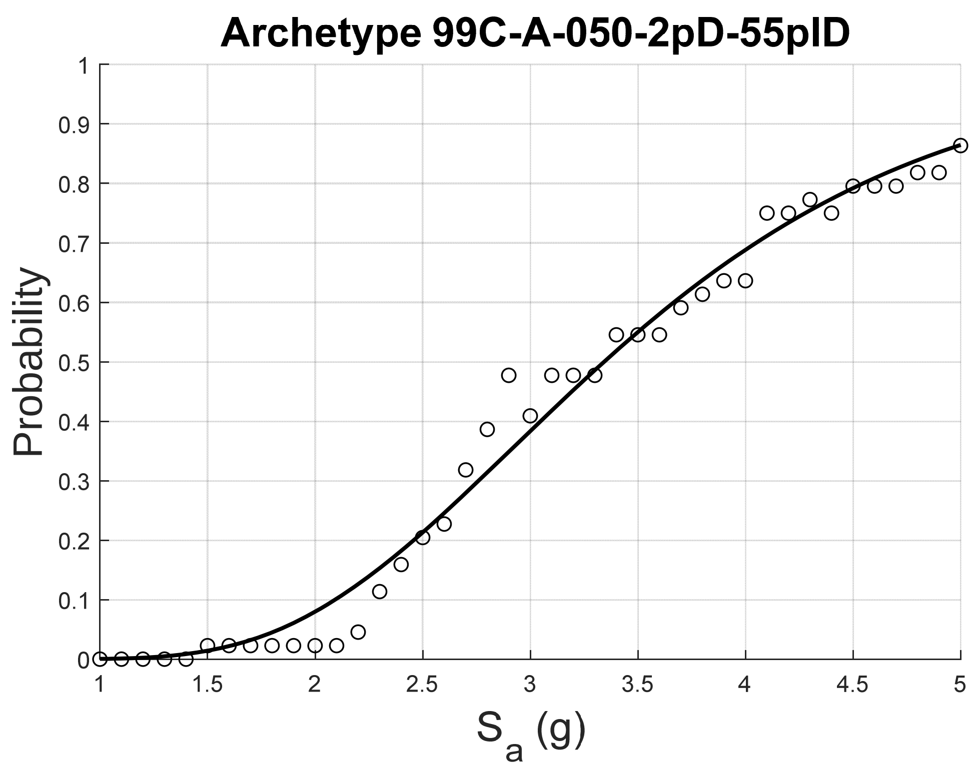


Figure G. 100. Archetype 99 collapse fragility
R=3, 2% damping, 5.5% interstory drift limit

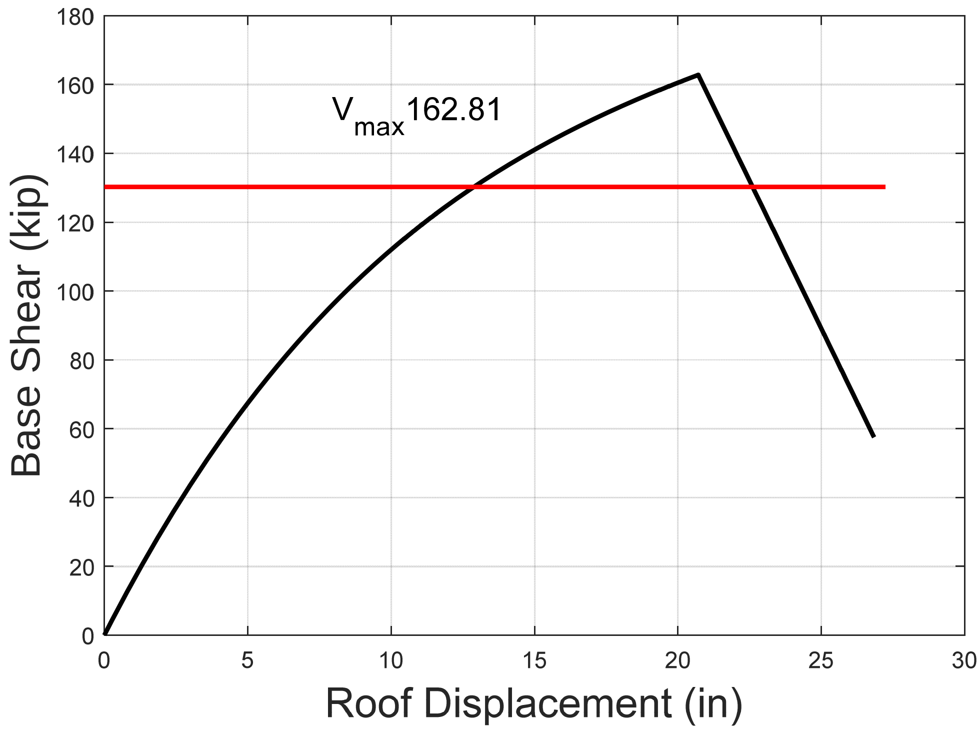


Figure G. 101. Archetype 100, R=3, static pushover

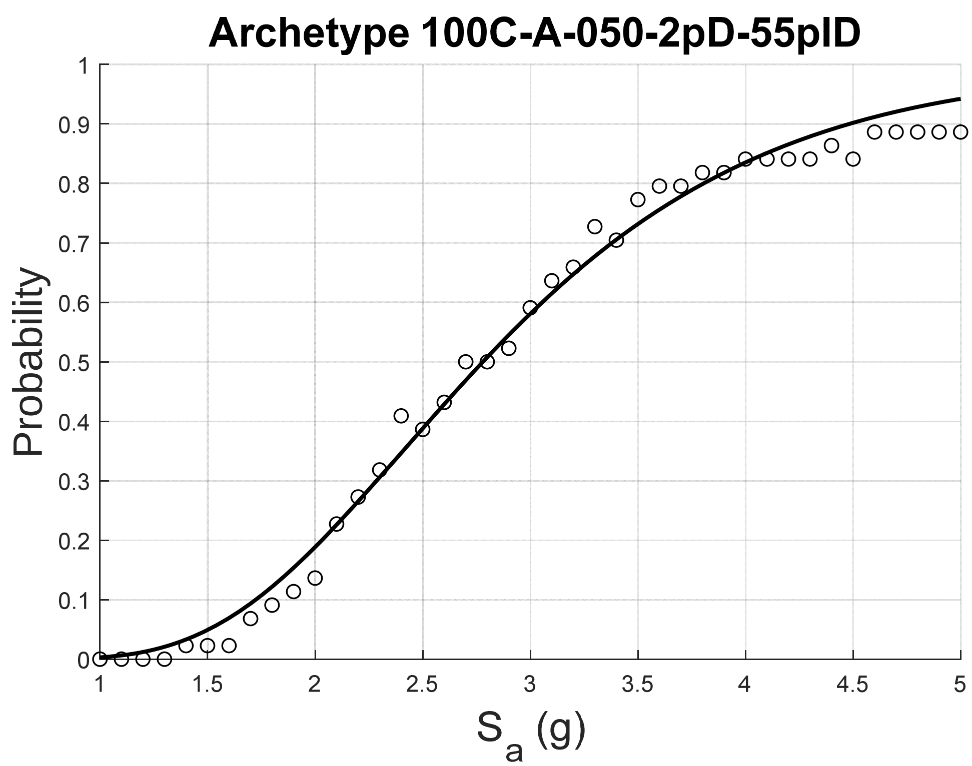


Figure G. 102. Archetype 100 collapse fragility
R=3, 2% damping, 5.5% interstory drift limit

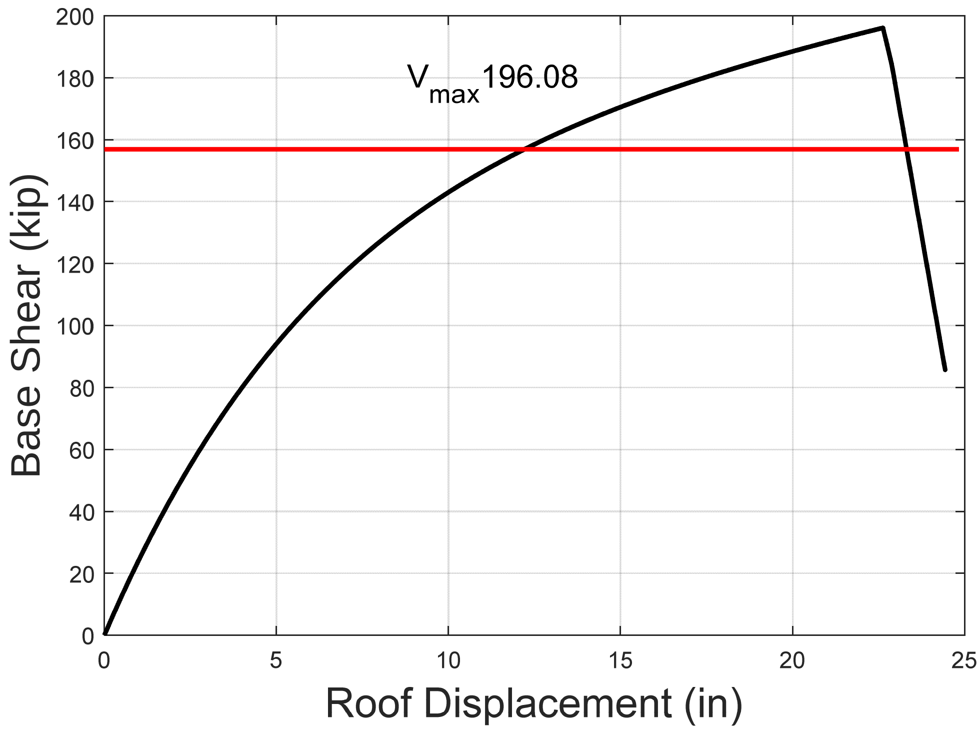


Figure G. 103. Archetype 101, R=3, static pushover

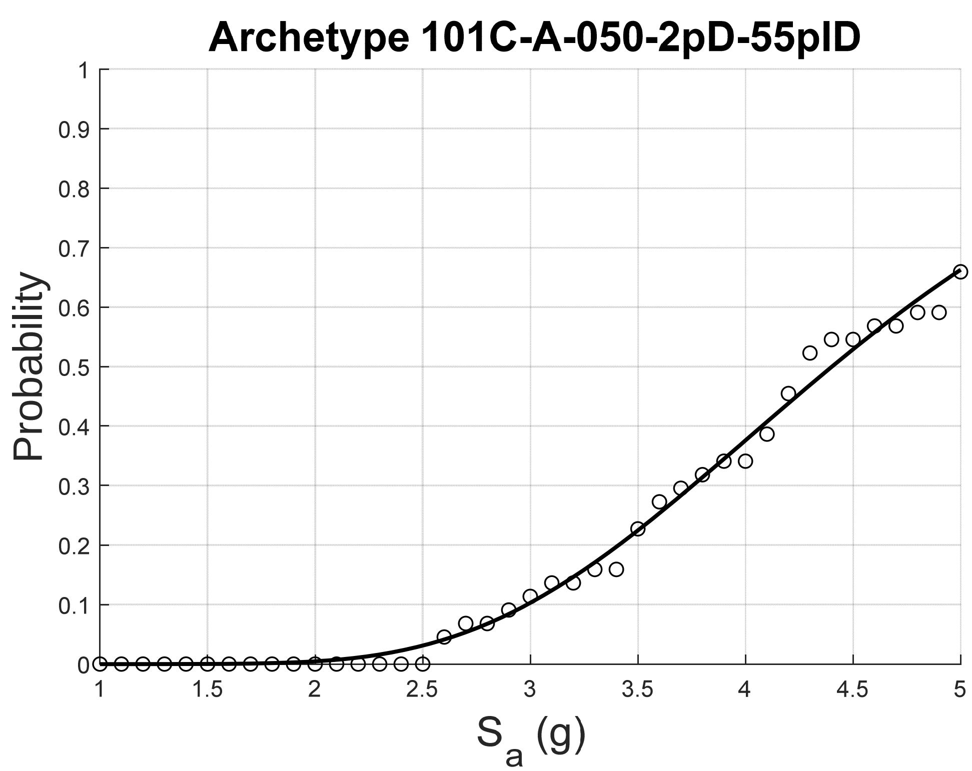


Figure G. 104. Archetype 101 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

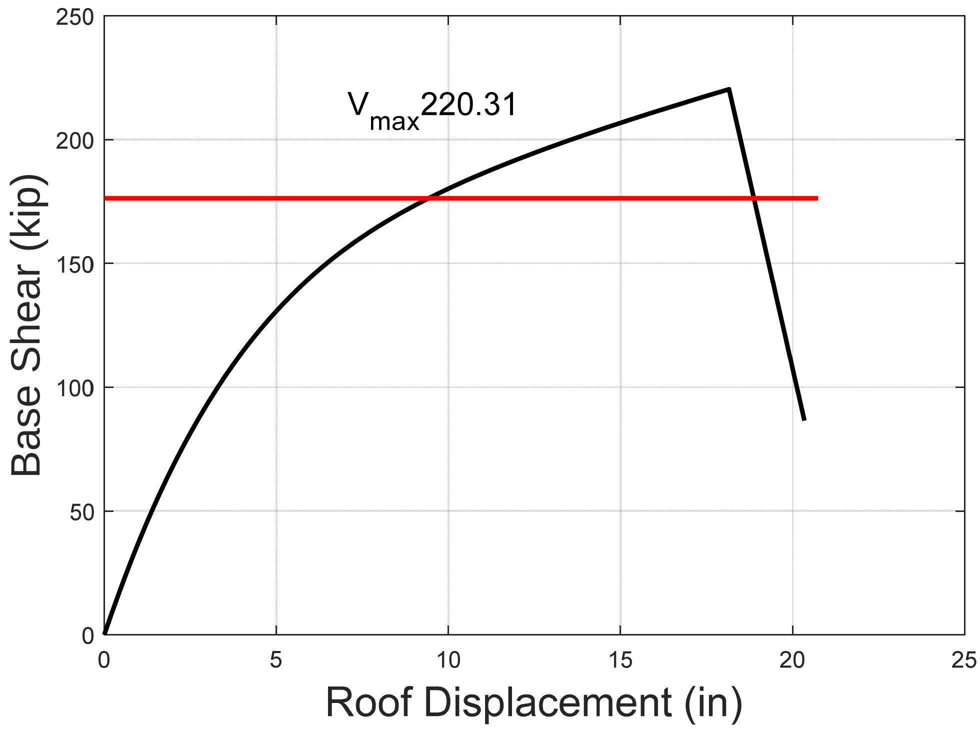


Figure G. 105. Archetype 102, R=3, static pushover

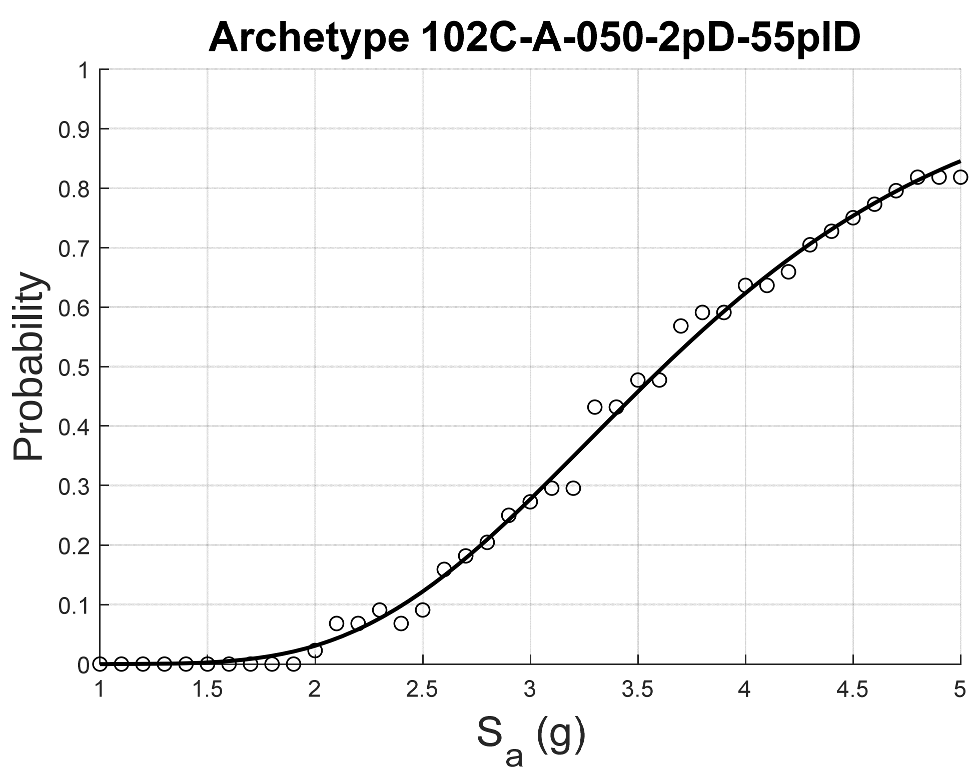


Figure G. 106. Archetype 102 collapse fragility
R=3, 2% damping, 5.5% interstory drift limit

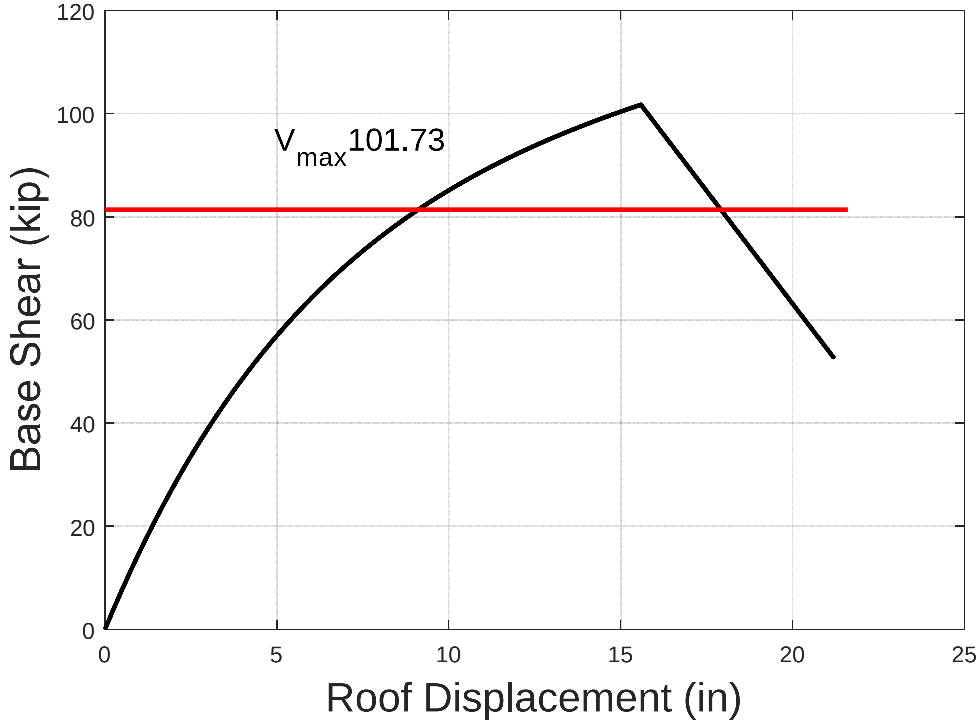


Figure G. 107. Archetype 109, R=3, static pushover

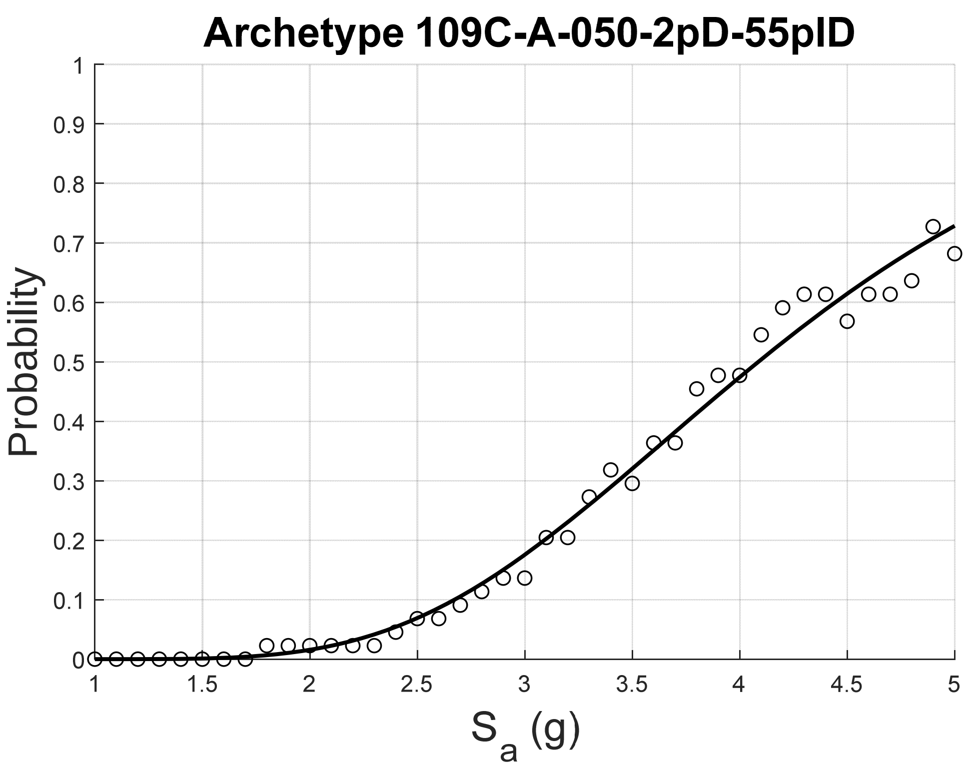


Figure G. 108. Archetype 109 collapse fragility
R=3, 2% damping, 5.5% interstory drift limit

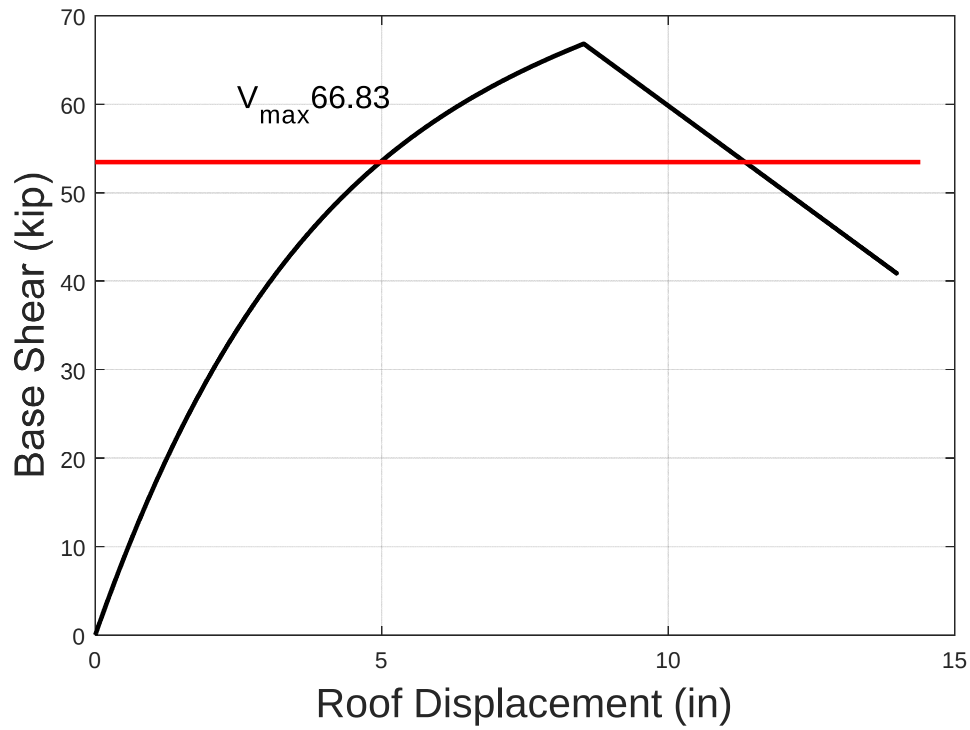


Figure G. 109. Archetype 110, R=3, static pushover

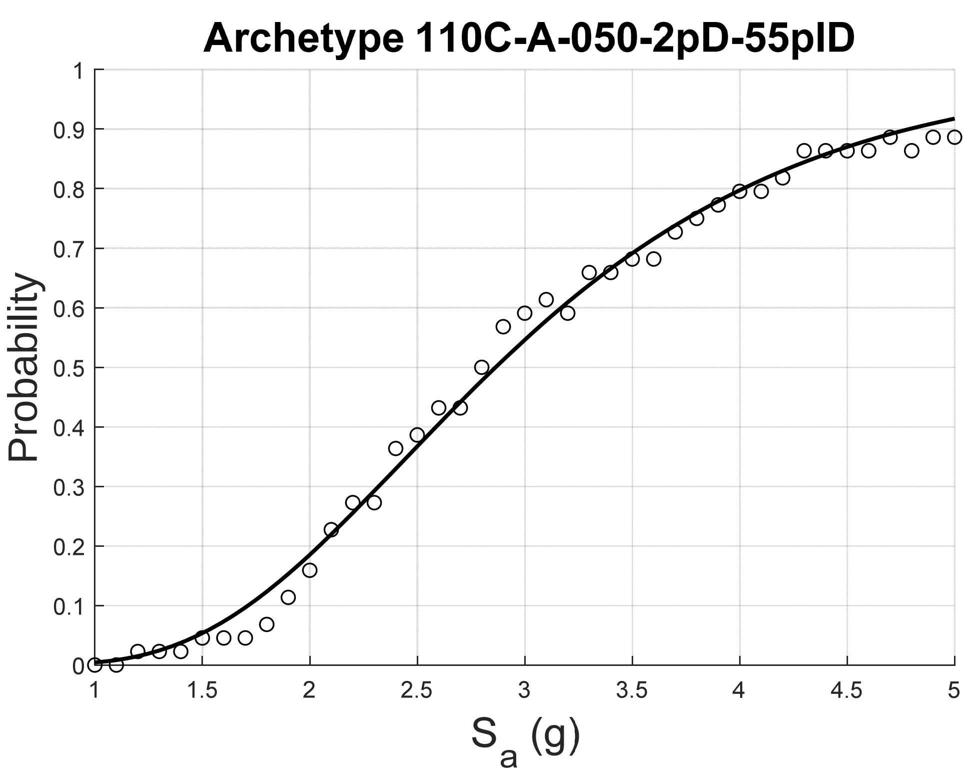


Figure G. 110. Archetype 110 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

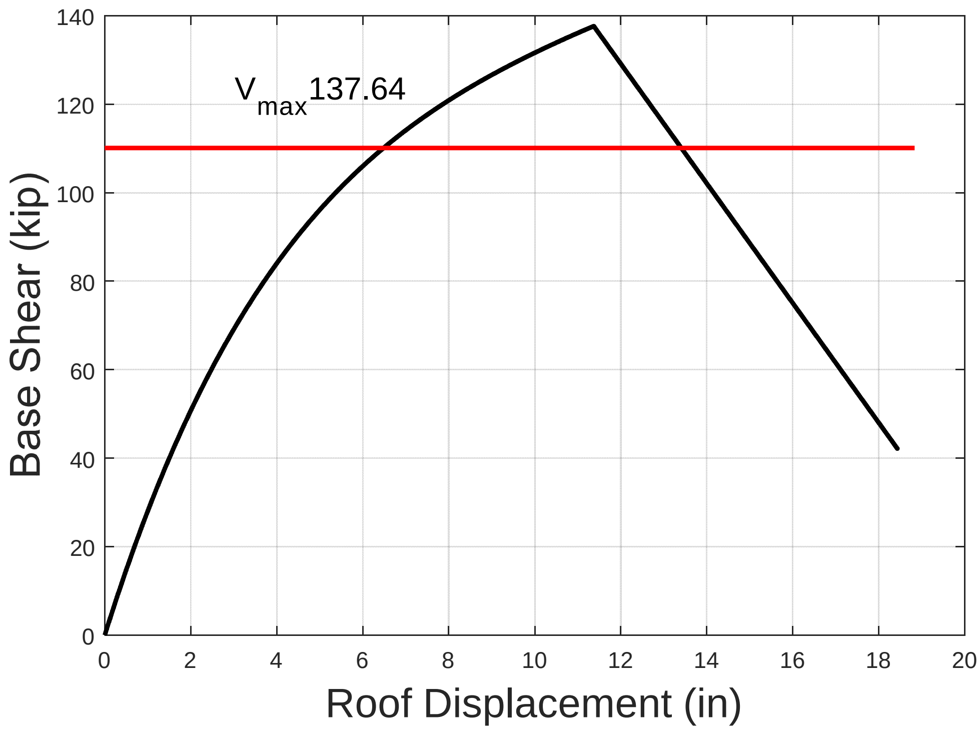


Figure G. 111. Archetype 111, R=3, static pushover

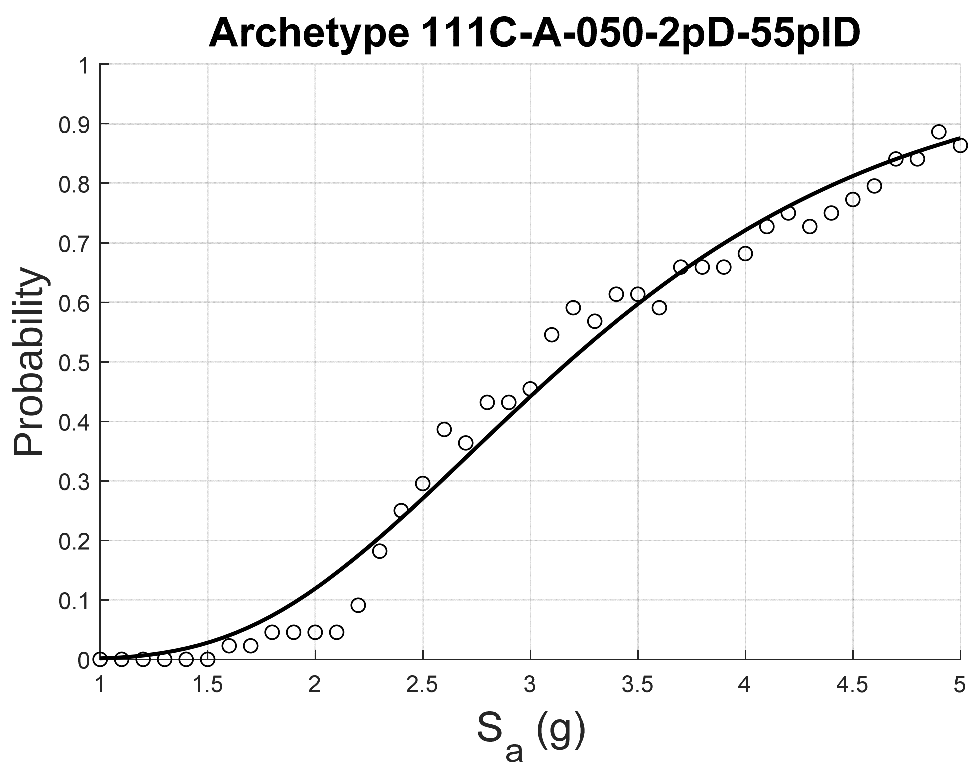


Figure G. 112. Archetype 111 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

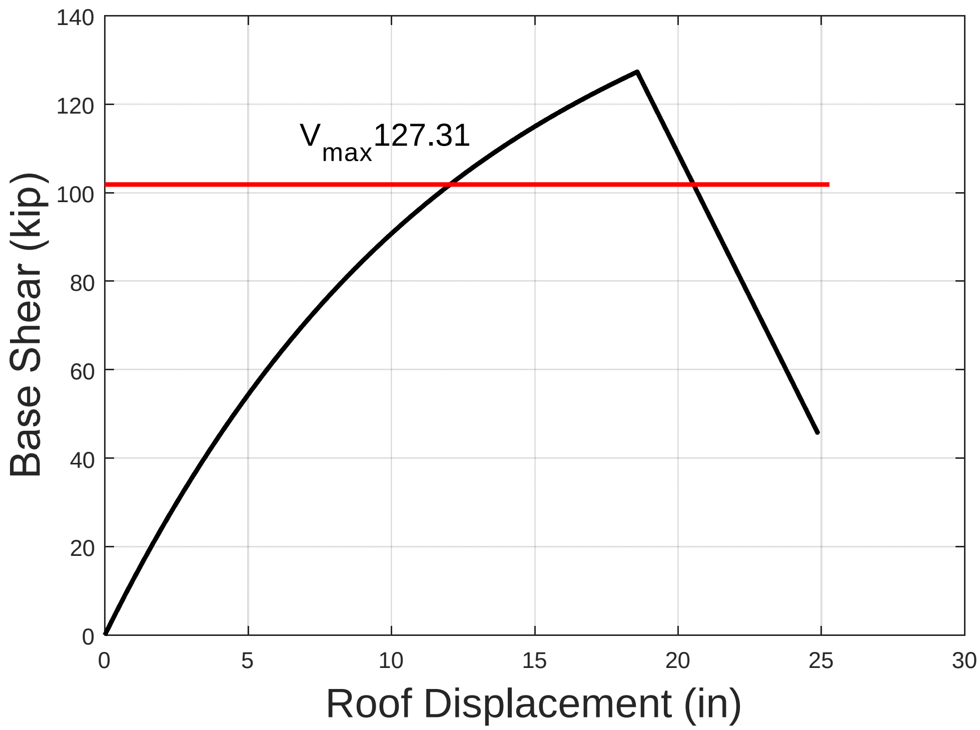


Figure G. 113. Archetype 112, R=3, static pushover

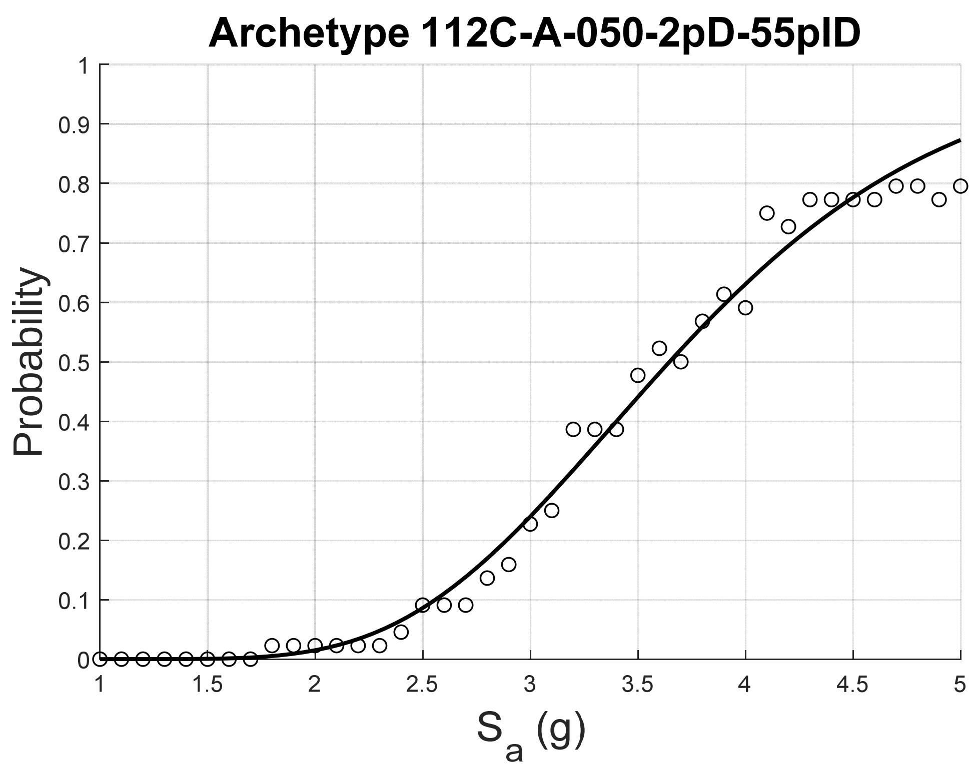


Figure G. 114. Archetype 112 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

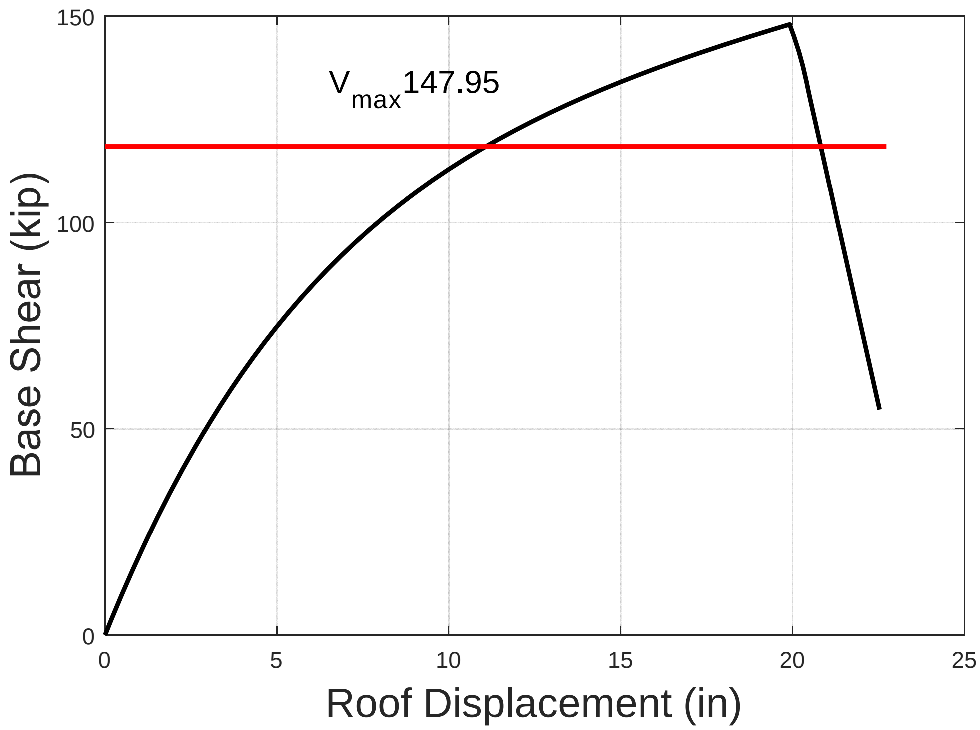


Figure G. 115. Archetype 113, R=3, static pushover

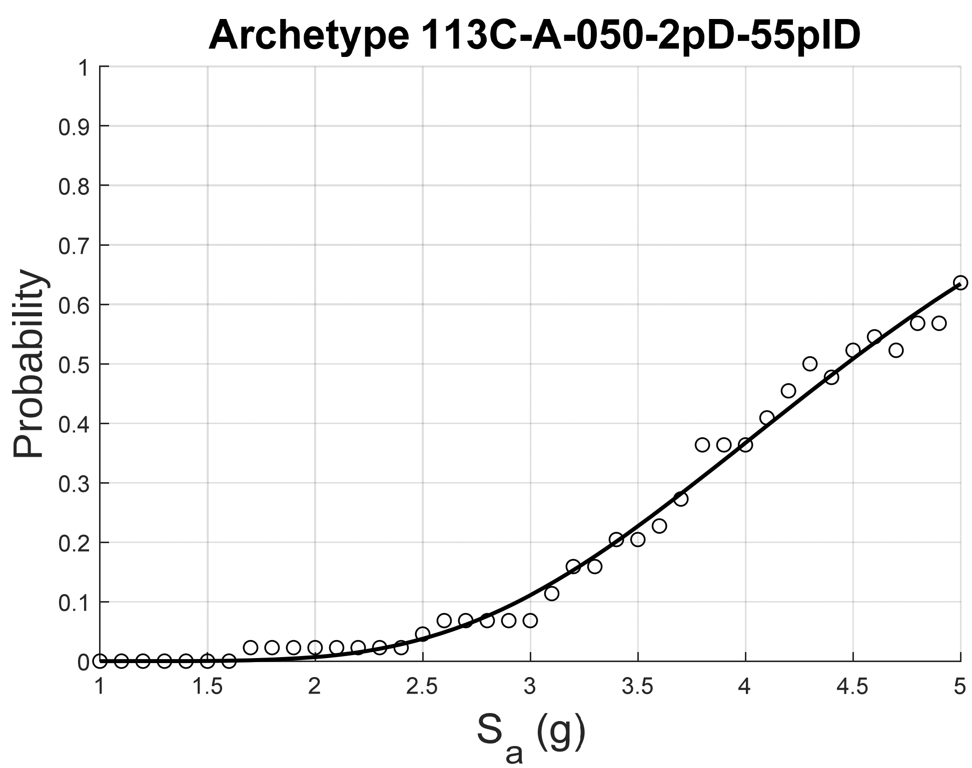


Figure G. 116. Archetype 113 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

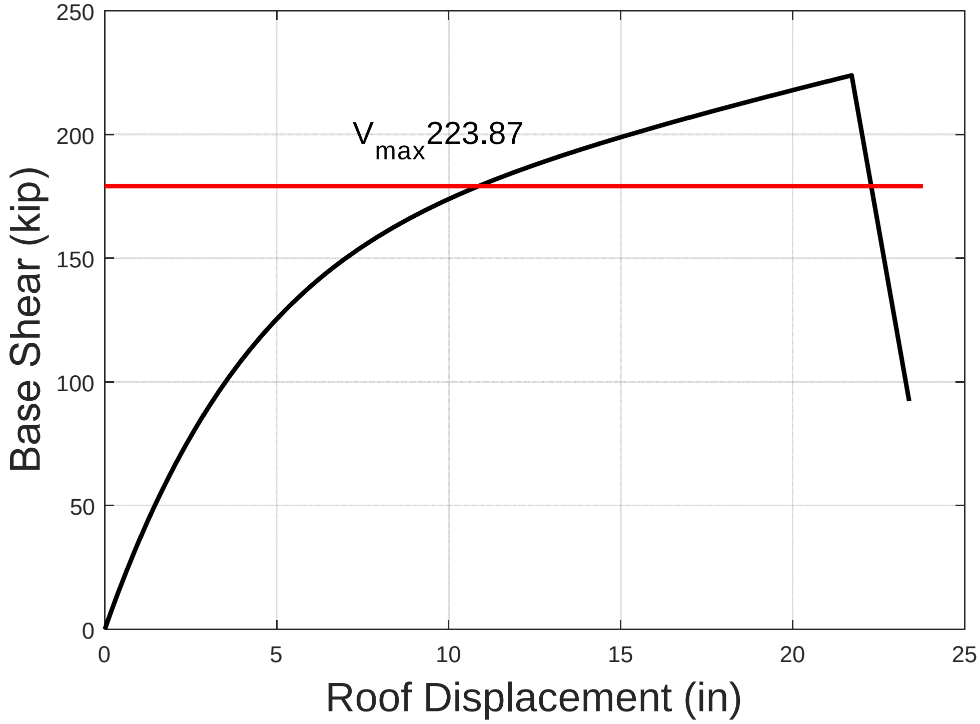


Figure G. 117. Archetype 114, R=3, static pushover

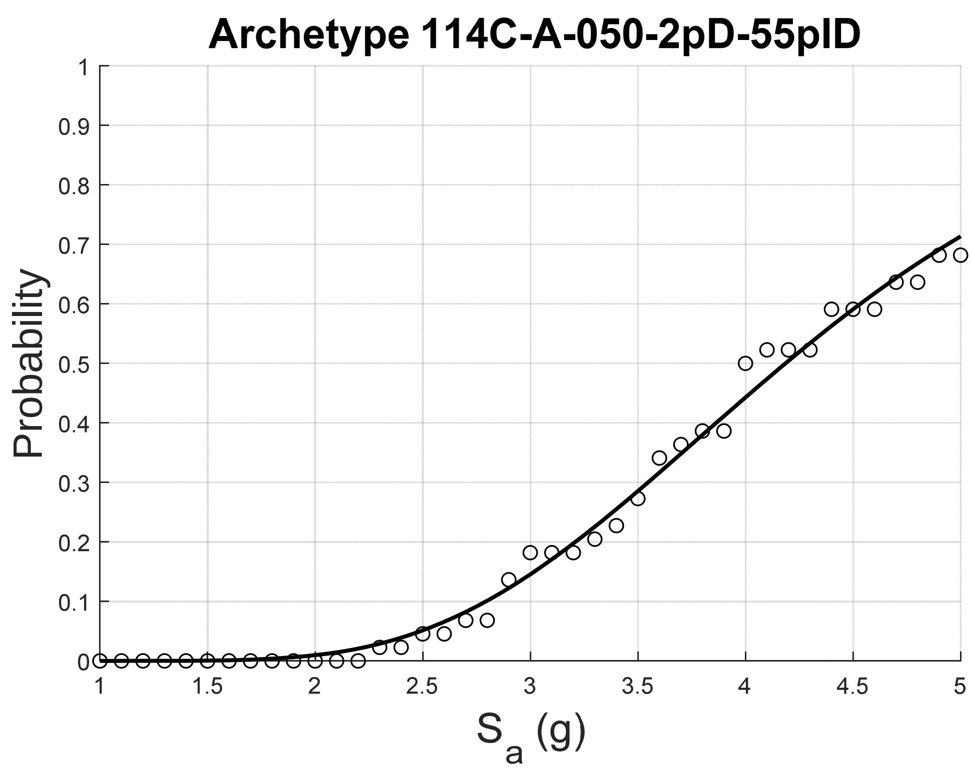


Figure G. 118. Archetype 114 collapse fragility R=3, 2% damping, 5.5% interstory drift limit

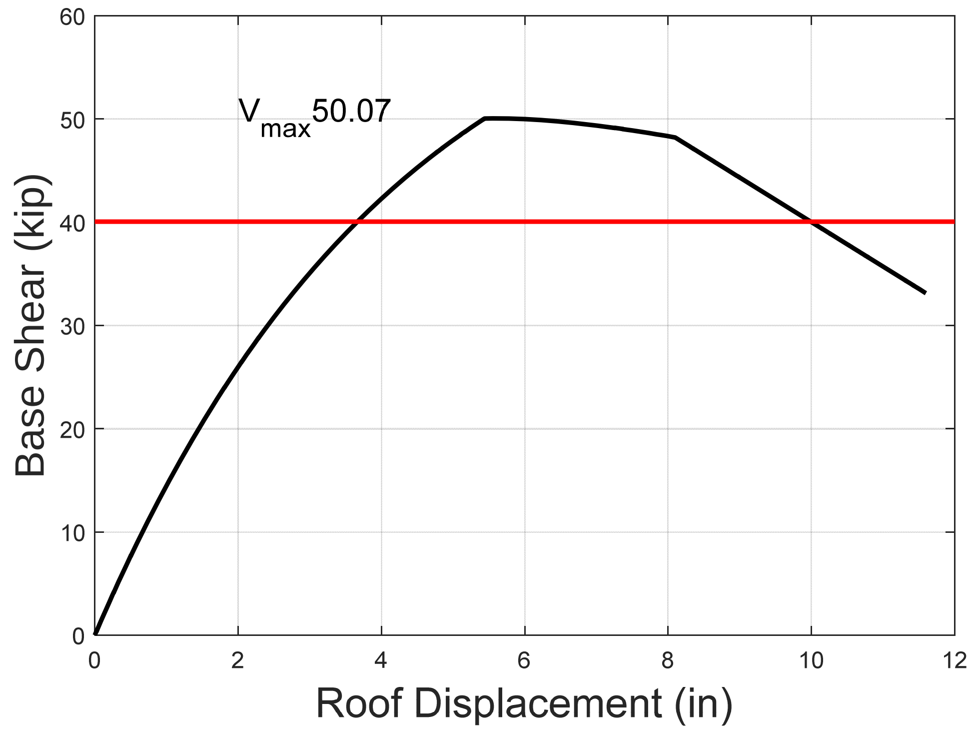


Figure G. 119. Archetype 121, R=3, static pushover

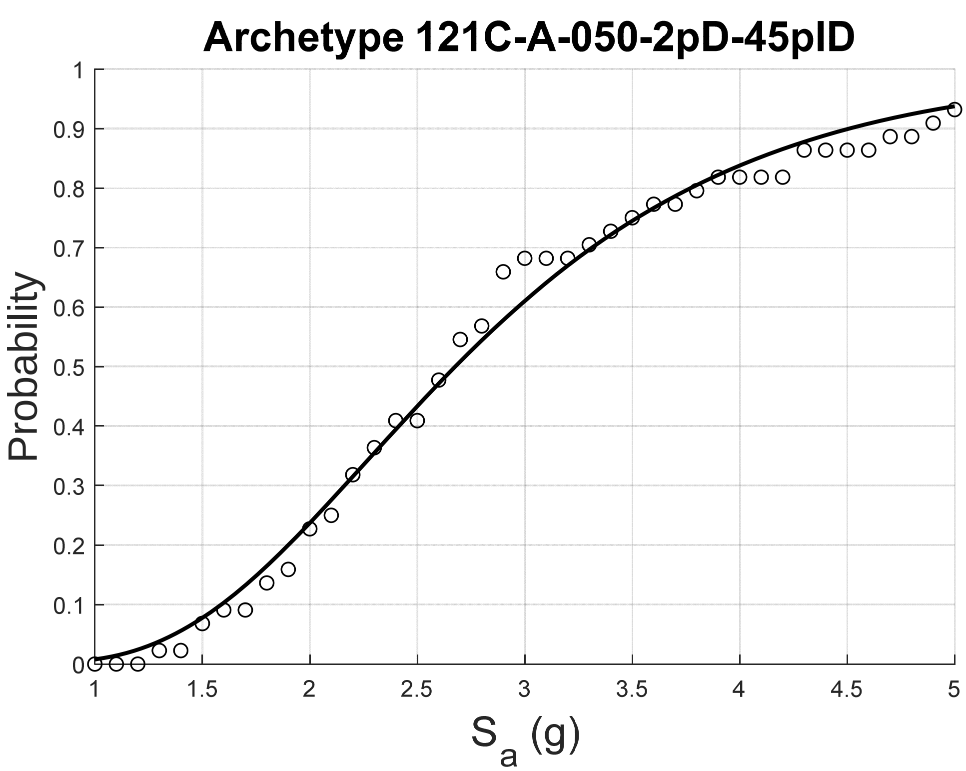


Figure G. 120. Archetype 121 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

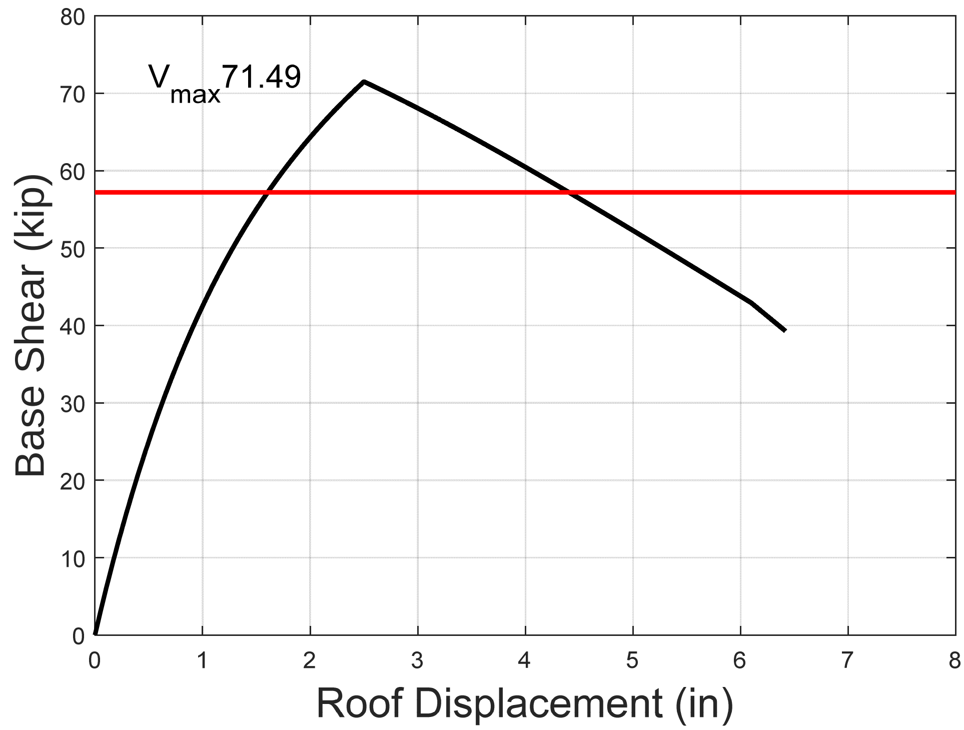


Figure G. 121. Archetype 122, R=3, static pushover

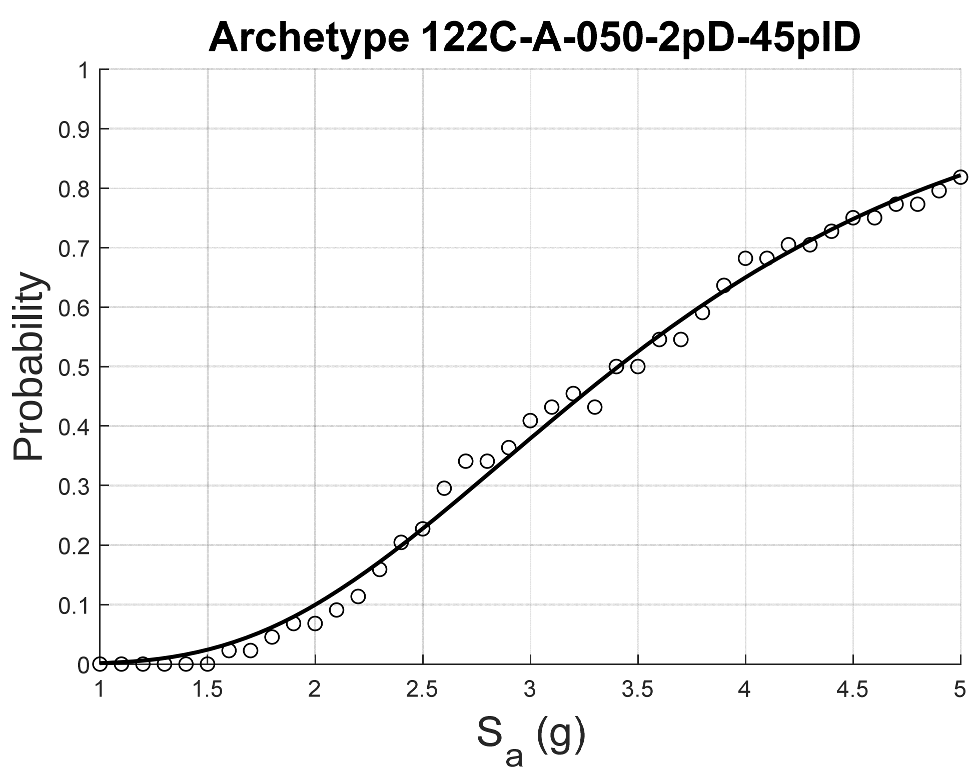


Figure G. 122. Archetype 122 collapse fragility
R=3, 2% damping, 4.5% interstory drift limit

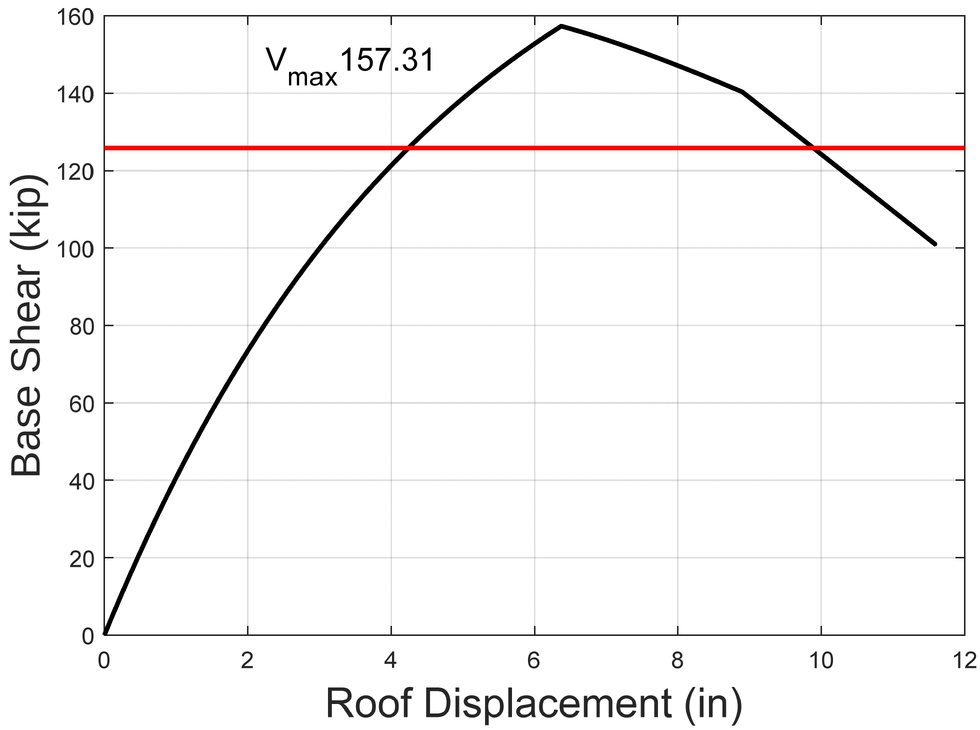


Figure G. 123. Archetype 123, R=3, static pushover

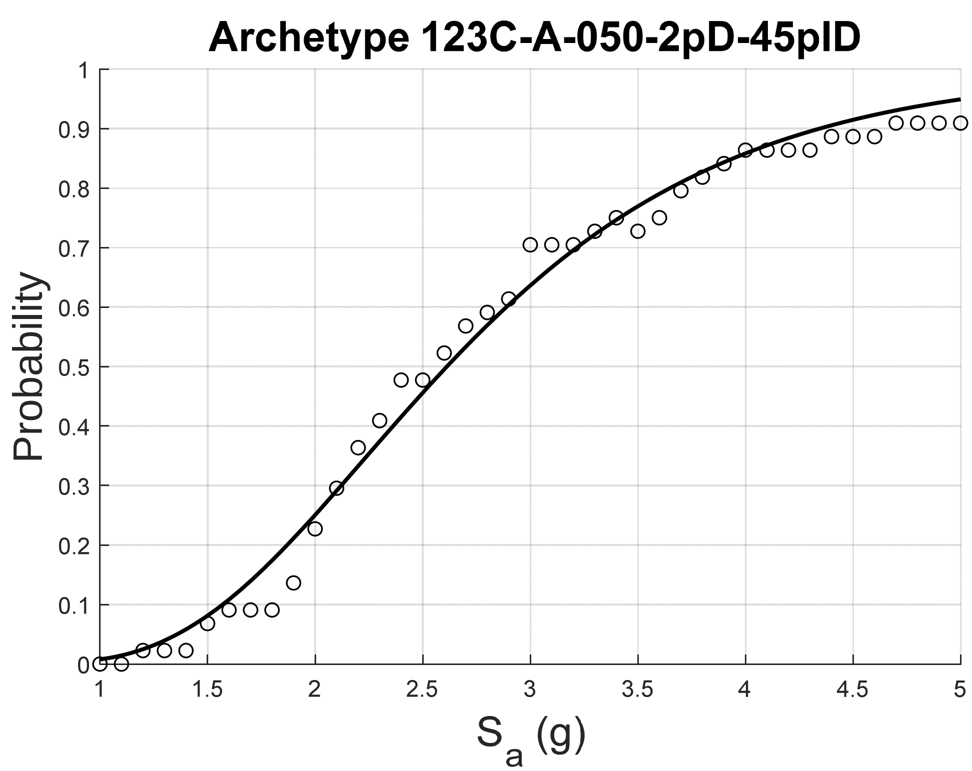


Figure G. 124. Archetype 123 collapse fragility
R=3, 2% damping, 4.5% interstory drift limit

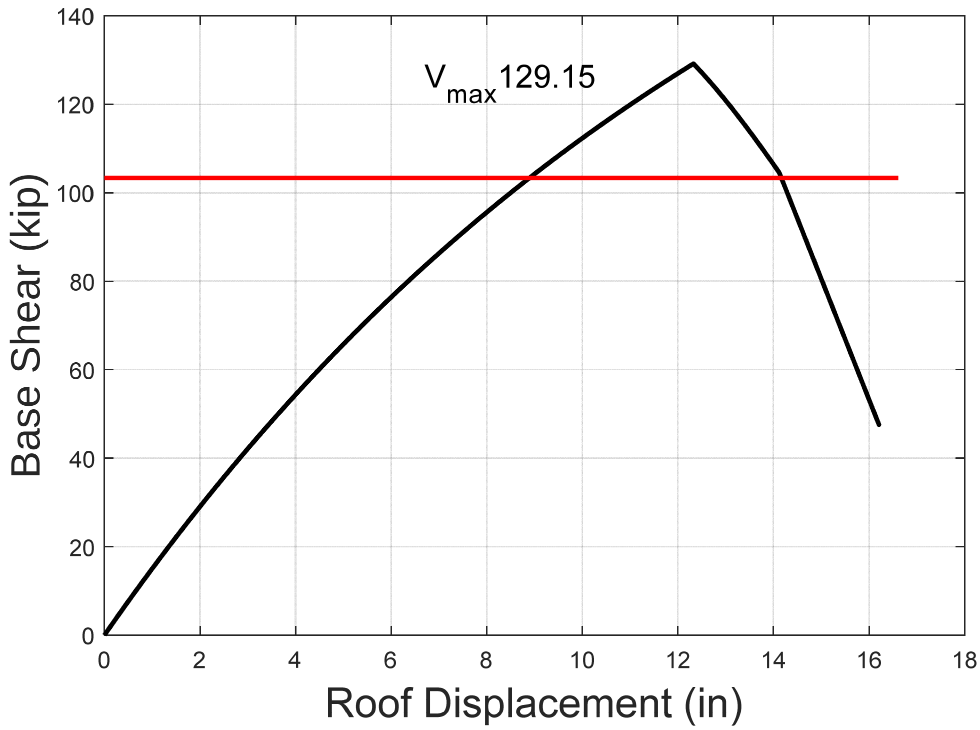


Figure G. 125. Archetype 124, R=3, static pushover

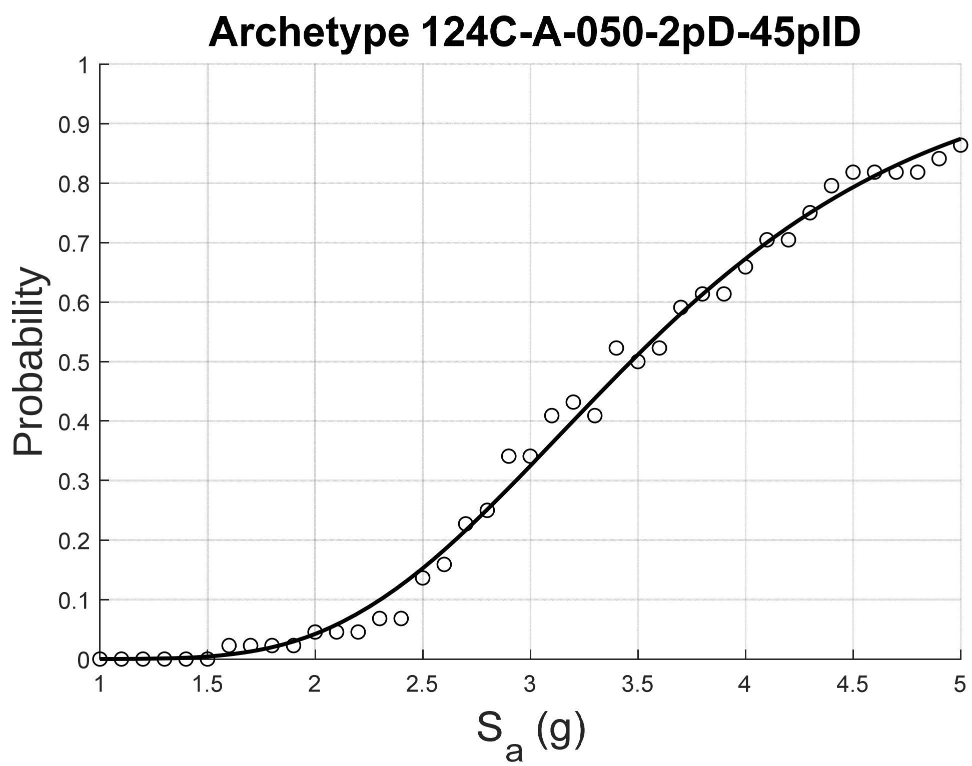


Figure G. 126. Archetype 124 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

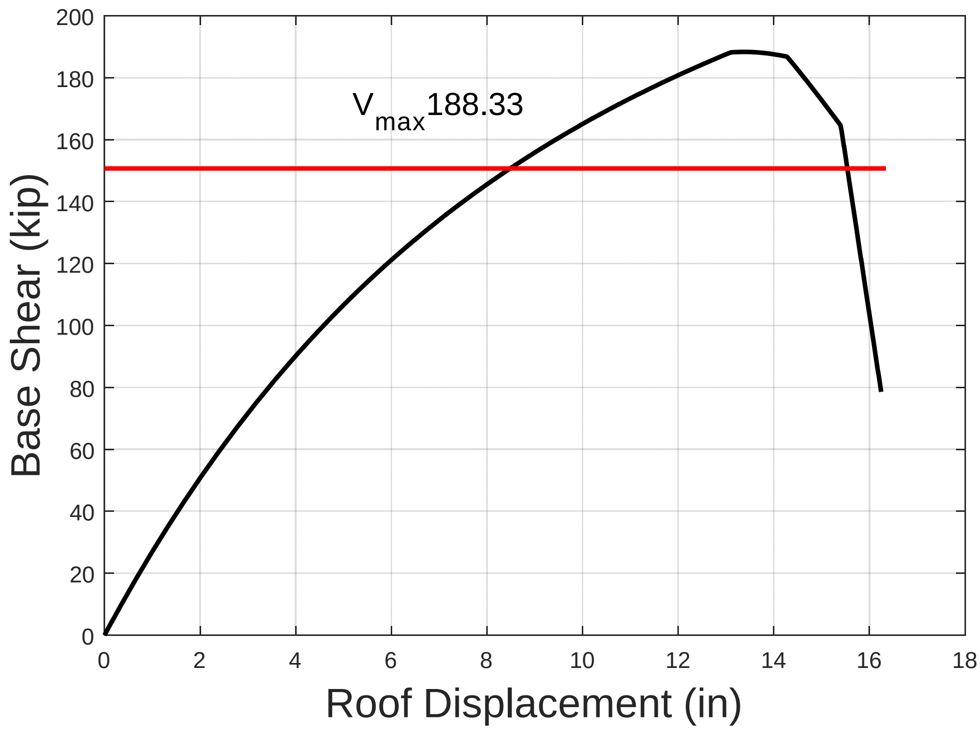


Figure G. 127. Archetype 125, R=3, static pushover

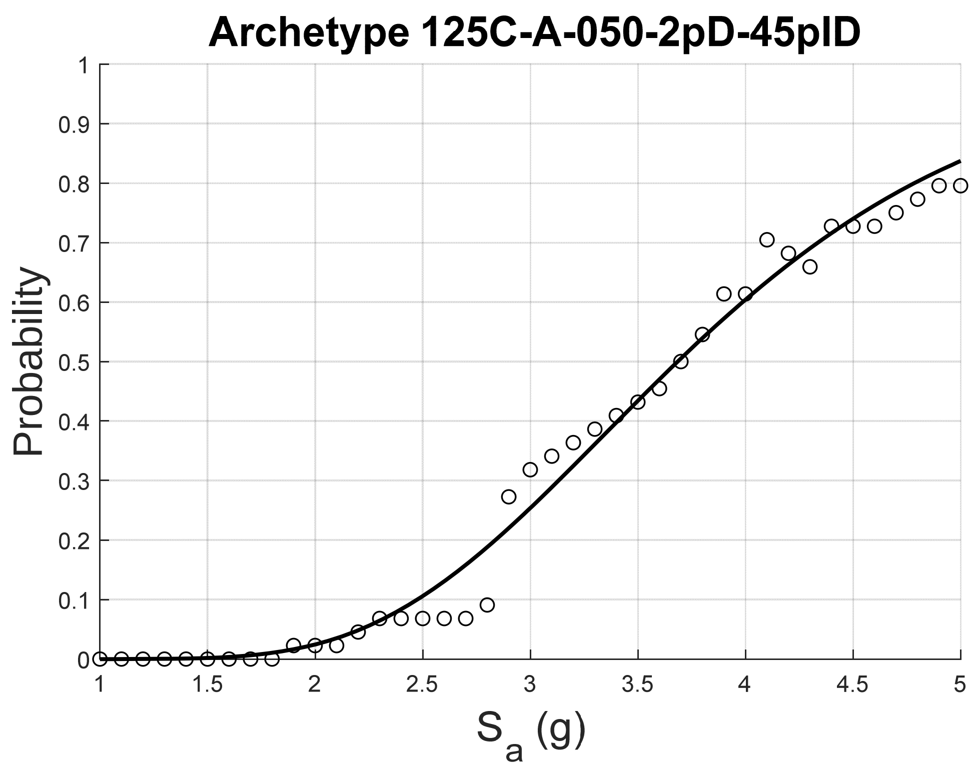


Figure G. 128. Archetype 125 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

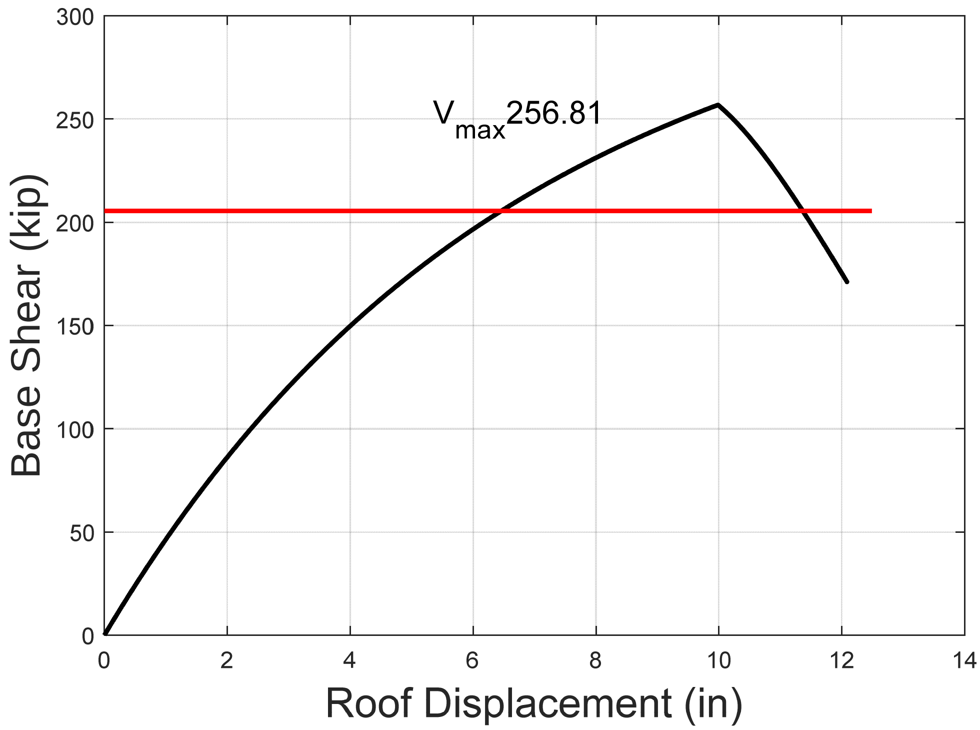


Figure G. 129. Archetype 126, R=3, static pushover

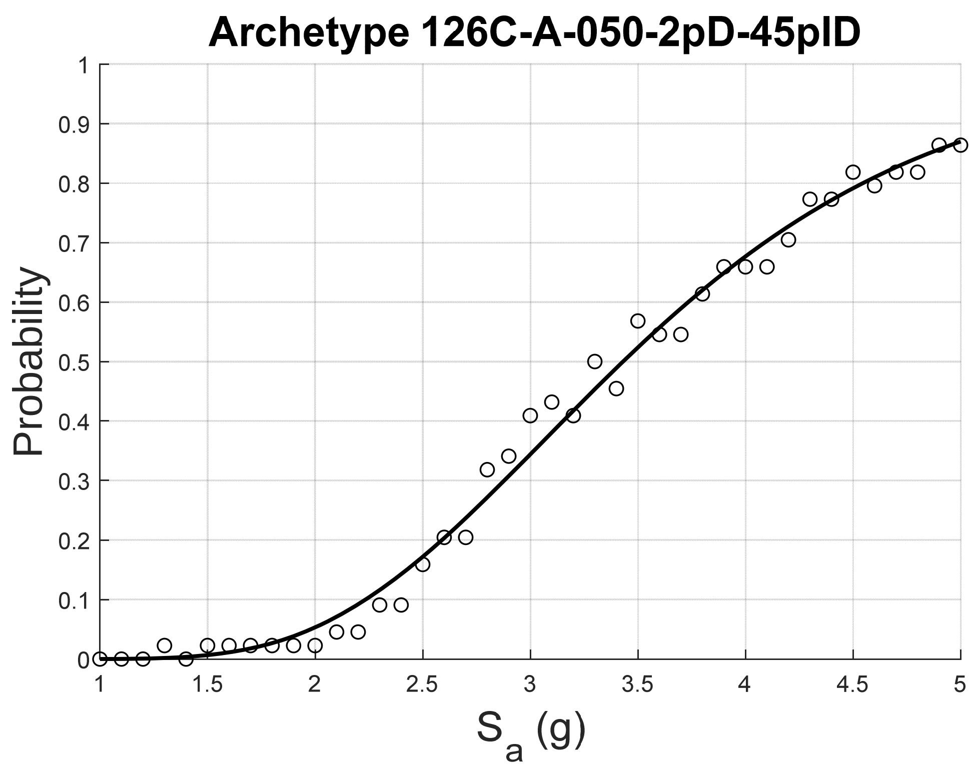


Figure G. 130. Archetype 126 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

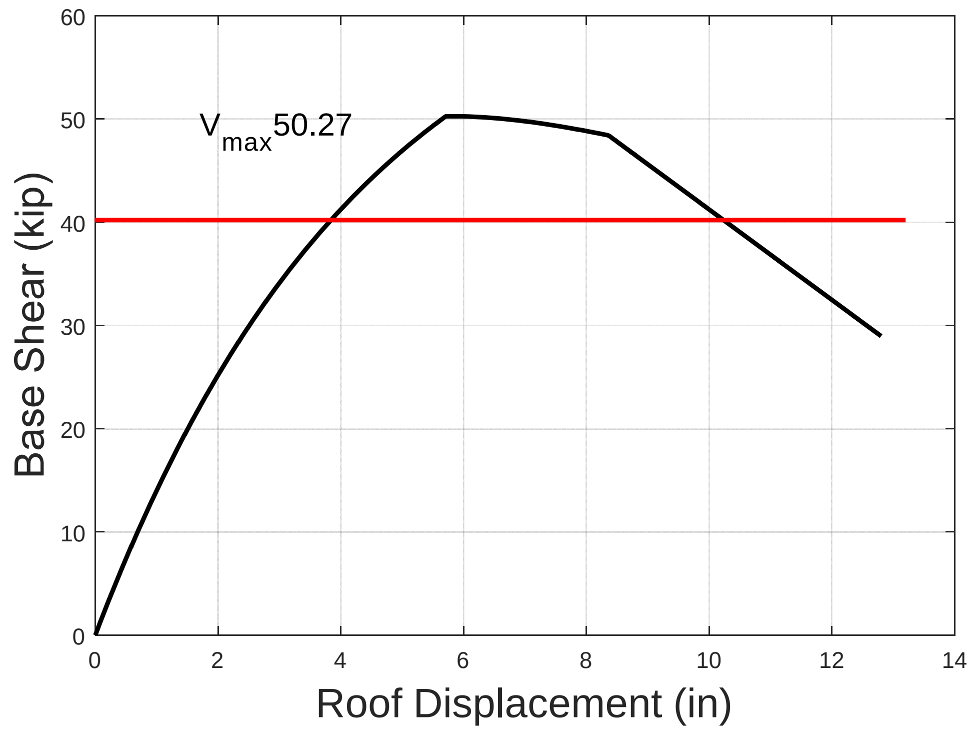


Figure G. 131. Archetype 133, R=3, static pushover

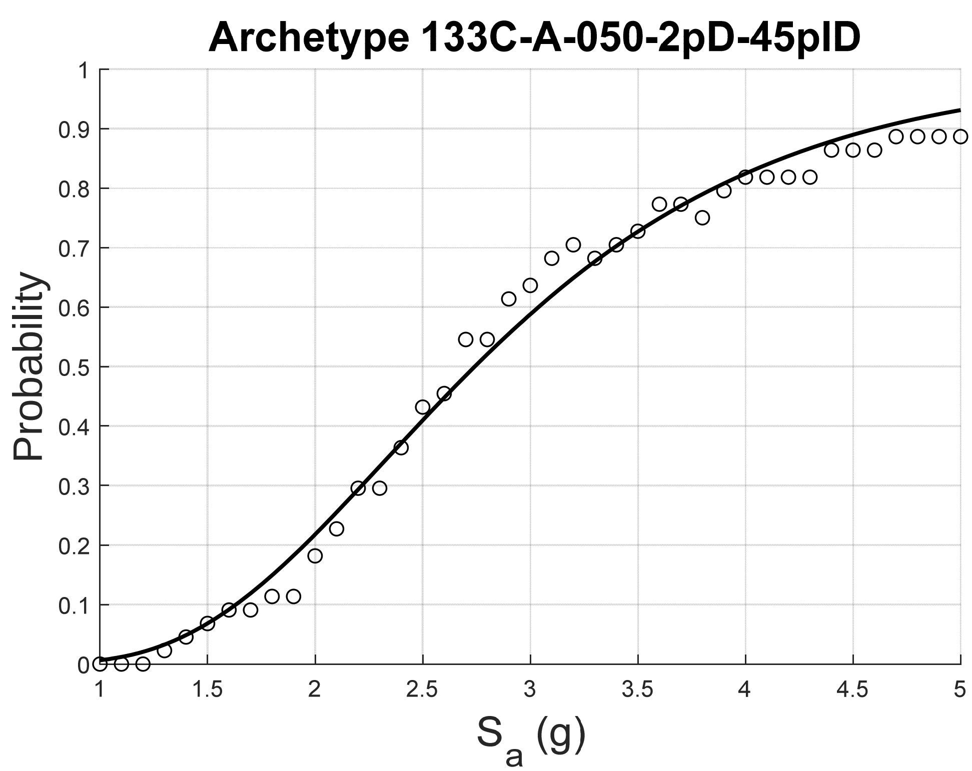


Figure G. 132. Archetype 133 collapse fragility
R=3, 2% damping, 4.5% interstory drift limit

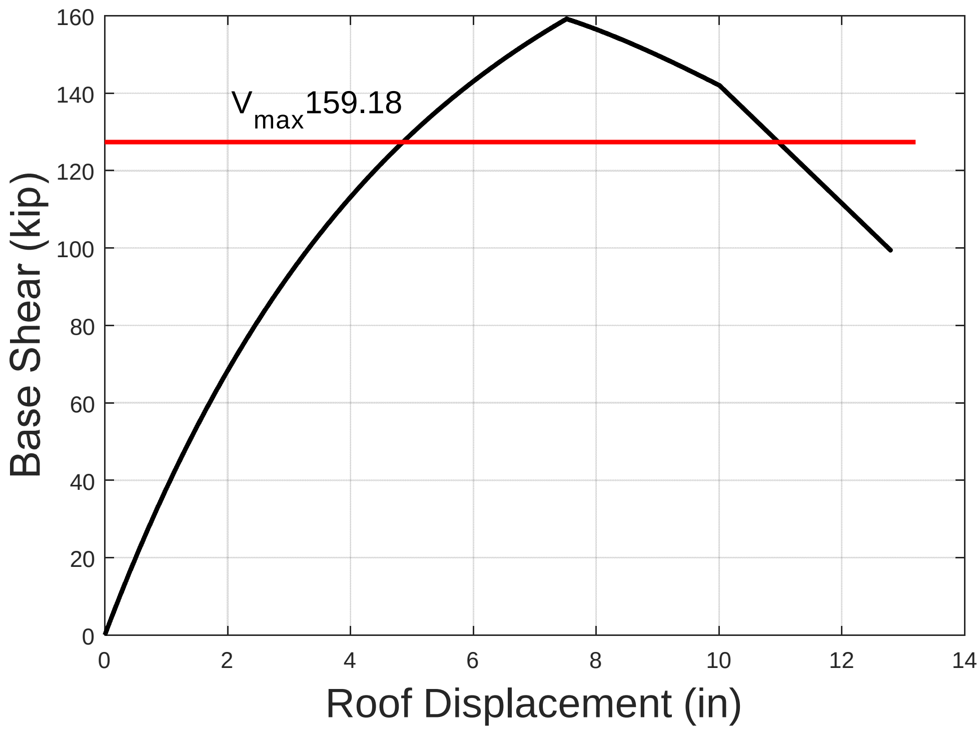


Figure G. 133. Archetype 135, R=3, static pushover

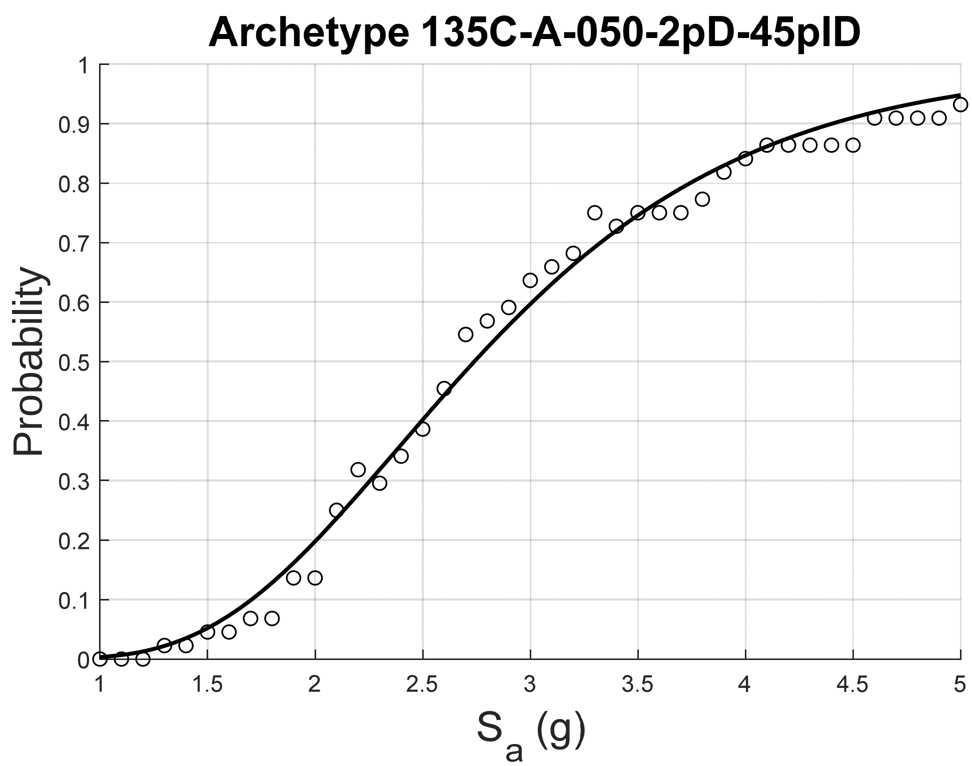


Figure G. 134. Archetype 135 collapse fragility
R=3, 2% damping, 4.5% interstory drift limit

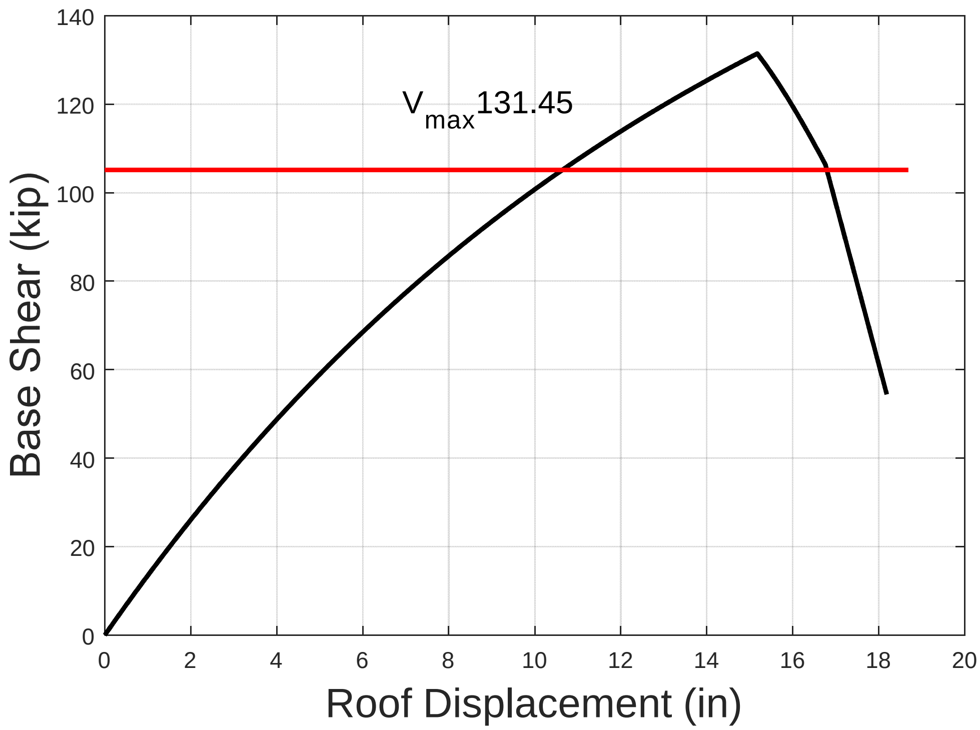


Figure G. 135. Archetype 136, R=3, static pushover

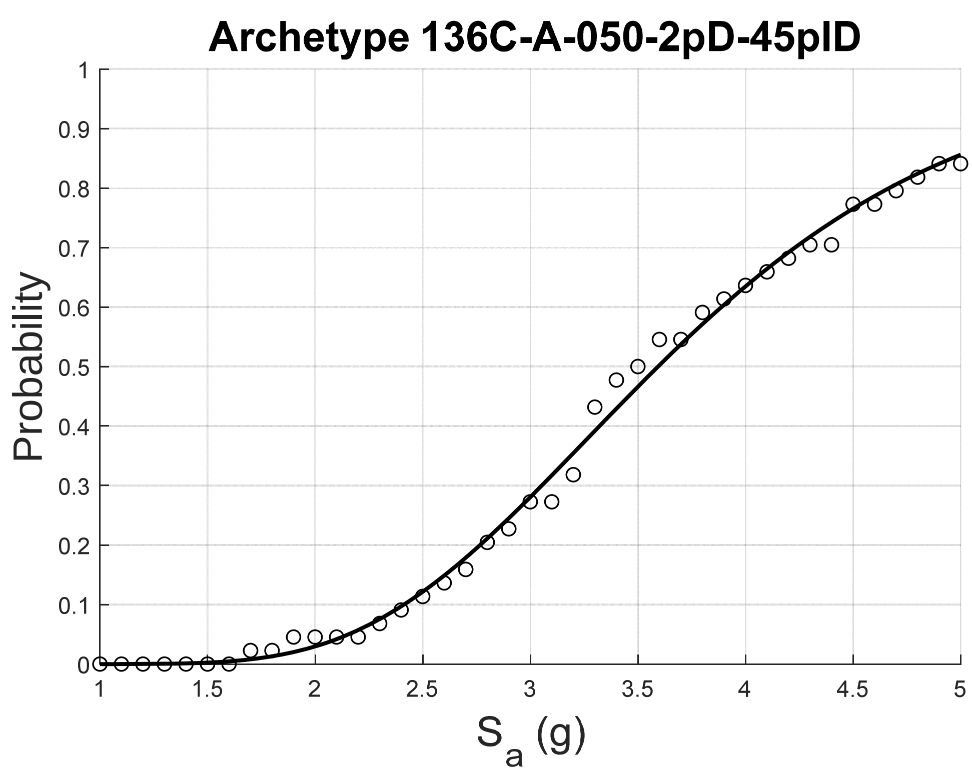


Figure G. 136. Archetype 136 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

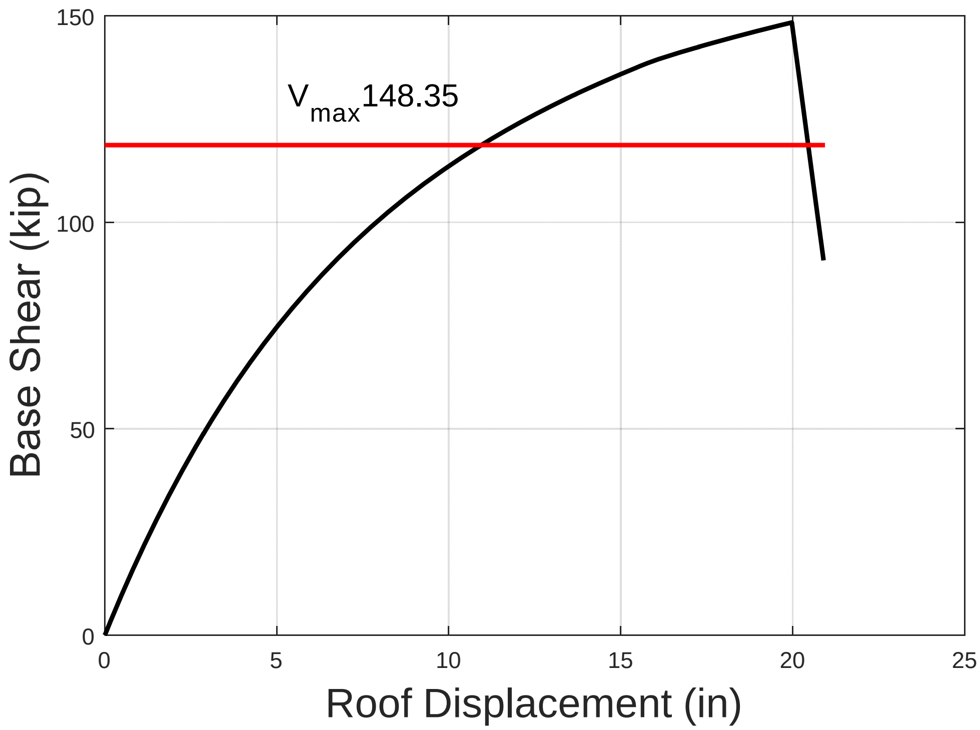


Figure G. 137. Archetype 137, R=3, static pushover

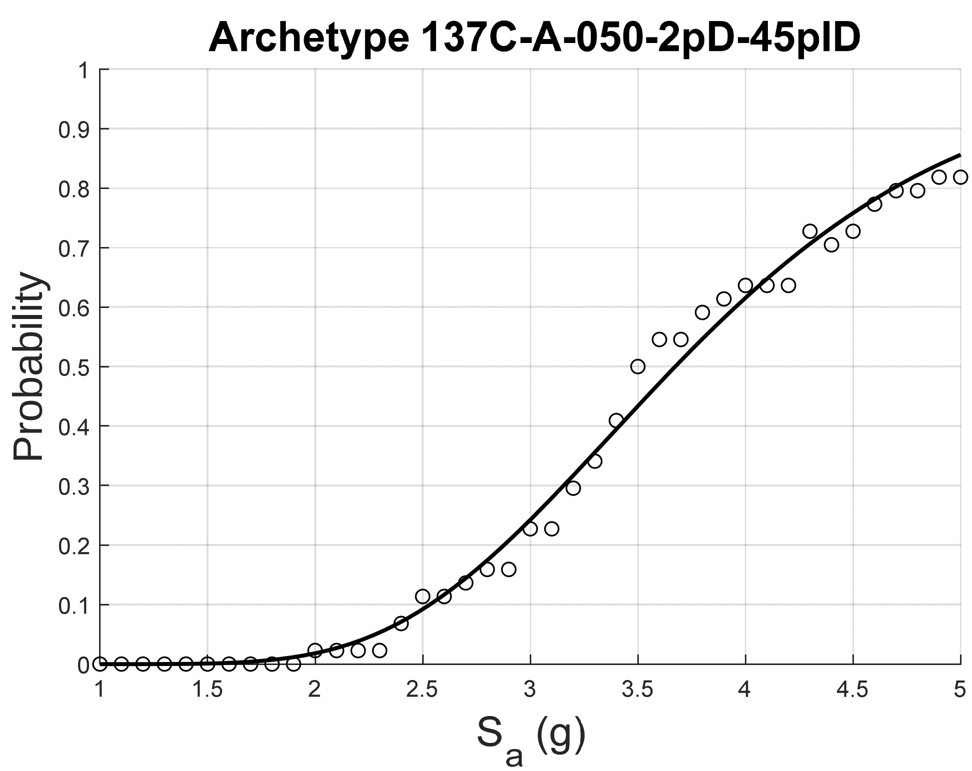


Figure G. 138. Archetype 137 collapse fragility R=3, 2% damping, 4.5% interstory drift limit

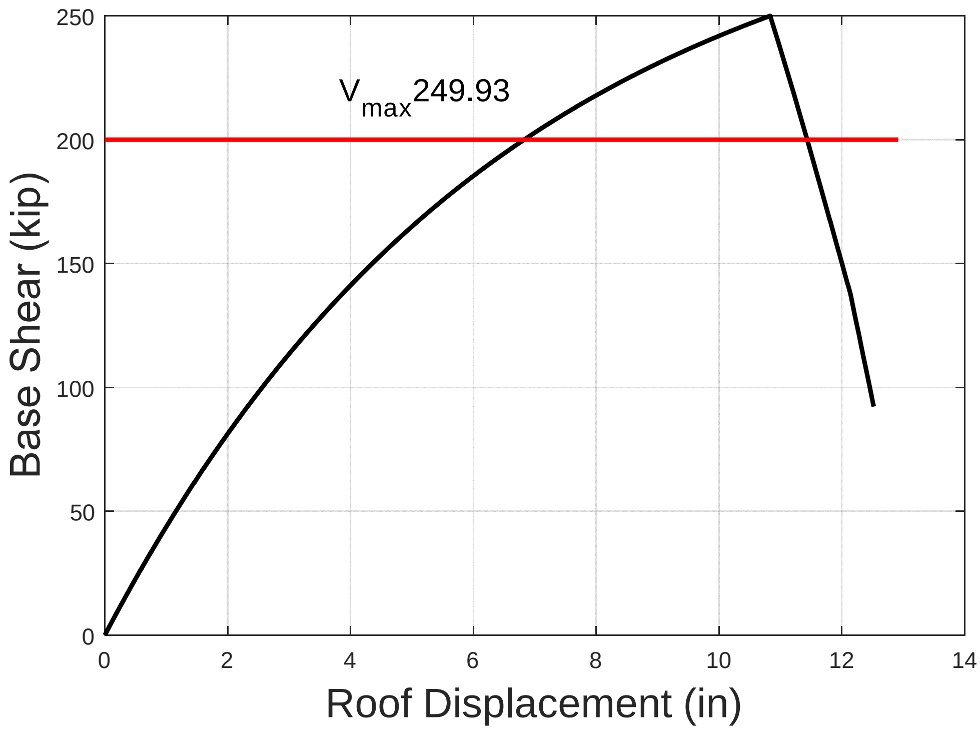


Figure G. 139. Archetype 138, R=3, static pushover

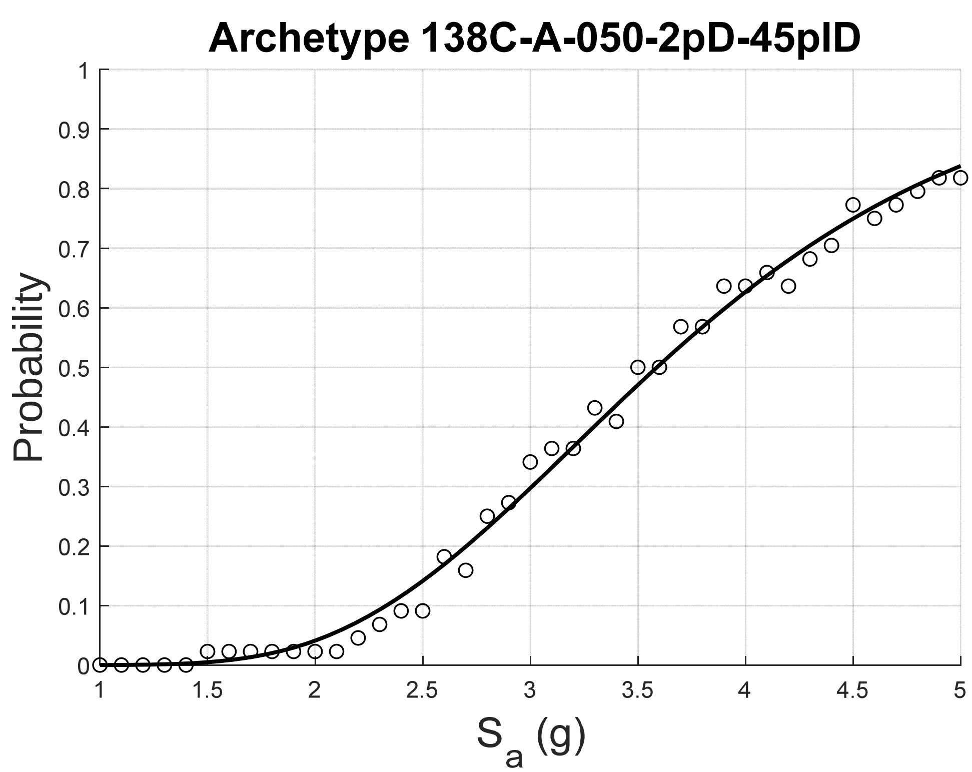


Figure G. 140. Archetype 138 collapse fragility
R=3, 2% damping, 4.5% interstory drift limit

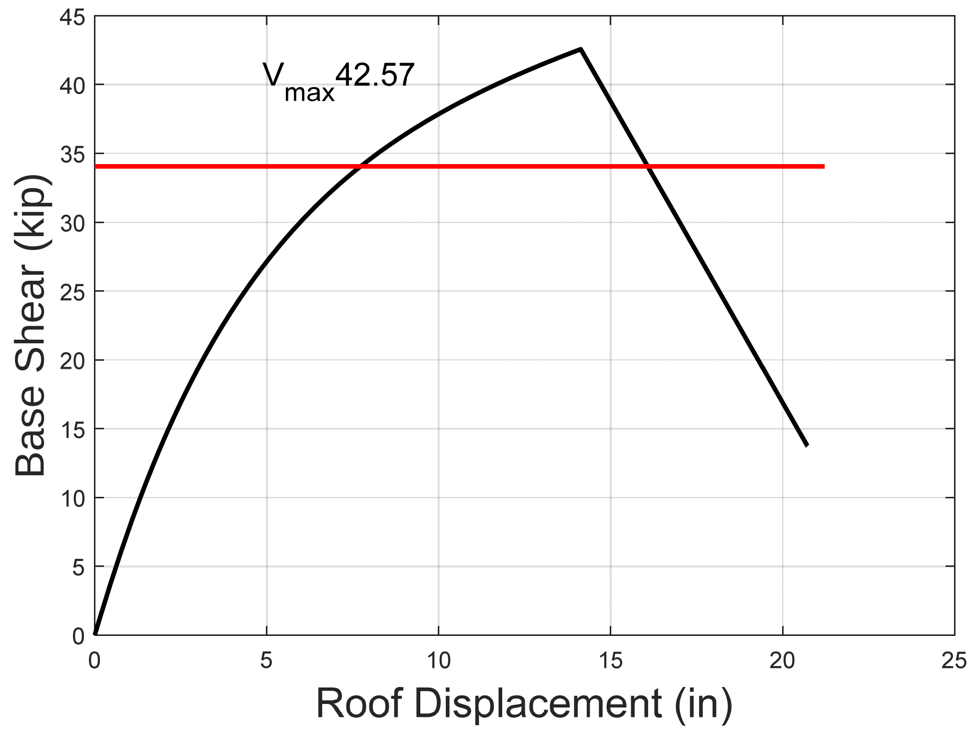


Figure G. 141. **Archetype 25, R=4, static pushover**

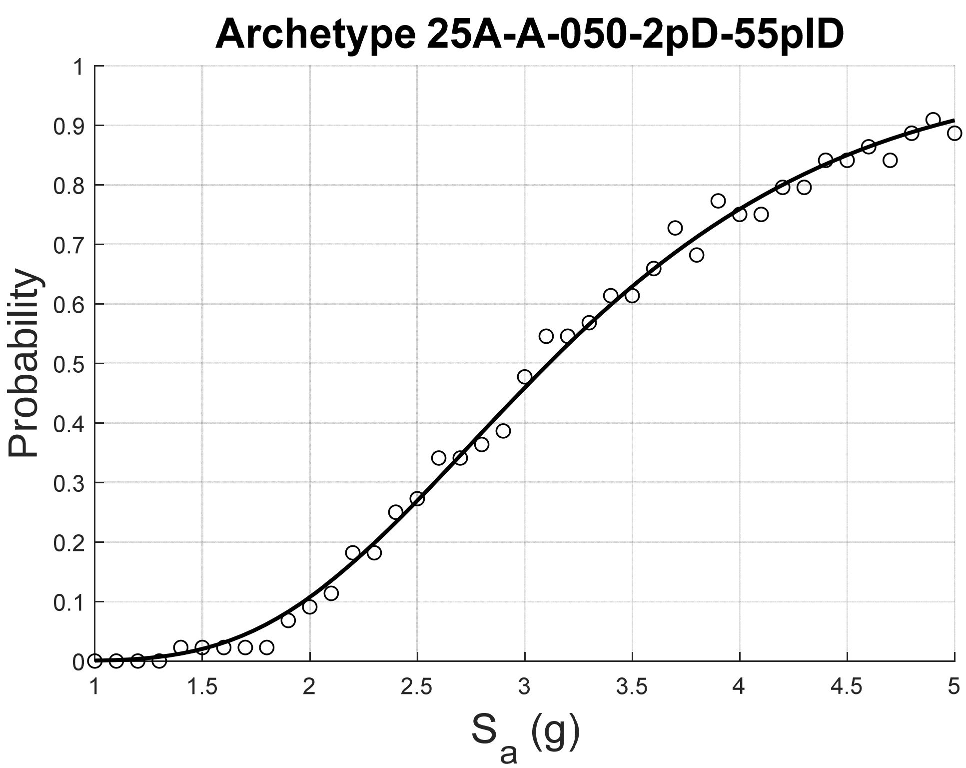


Figure G. 142. Archetype 25 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

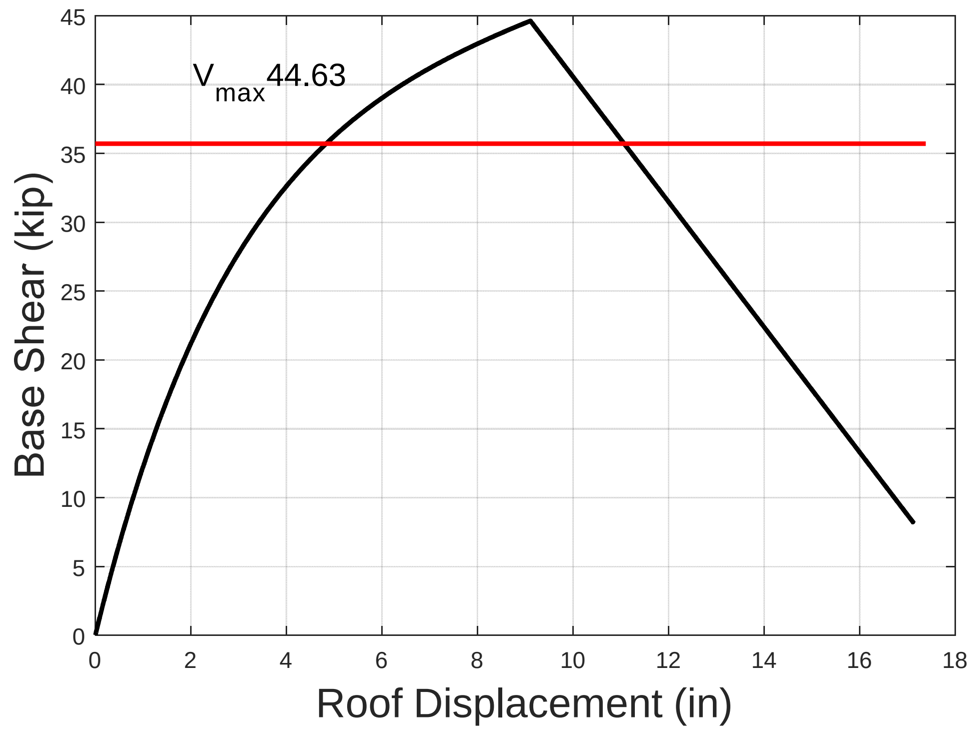


Figure G. 143. Archetype 26, R=4, static pushover

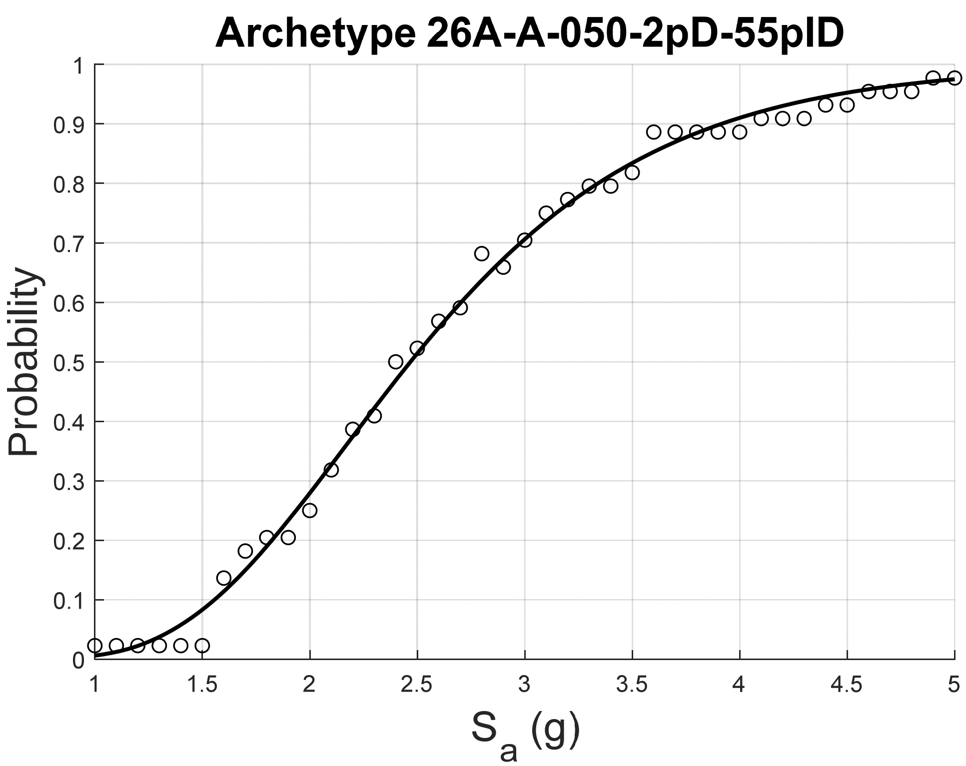


Figure G. 144. Archetype 26 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

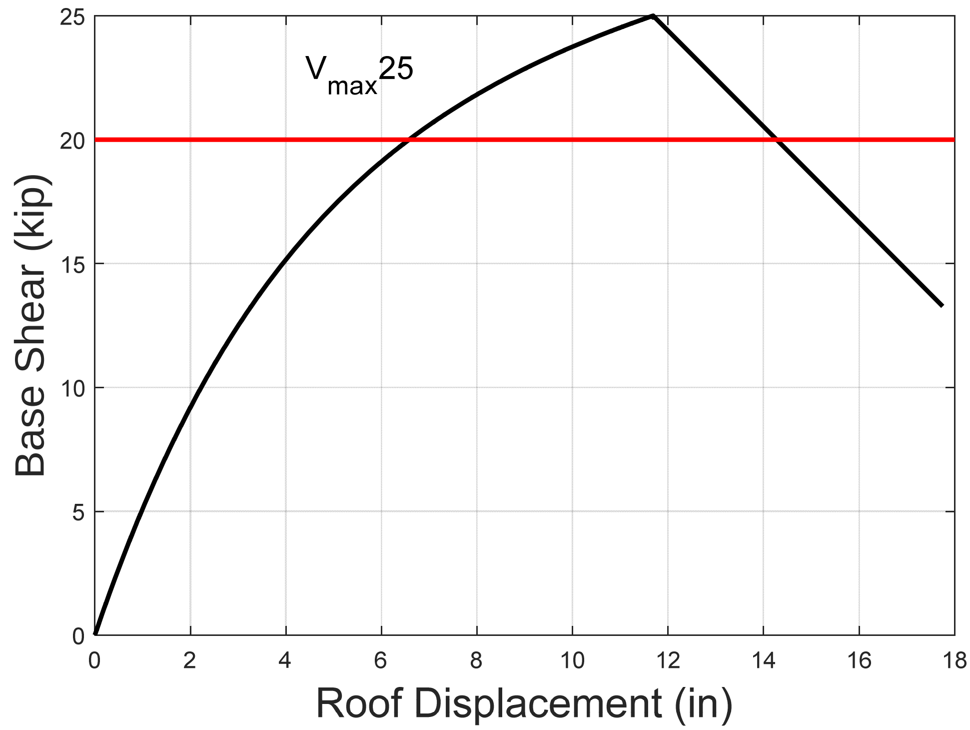


Figure G. 145. Archetype 27, R=4, static pushover

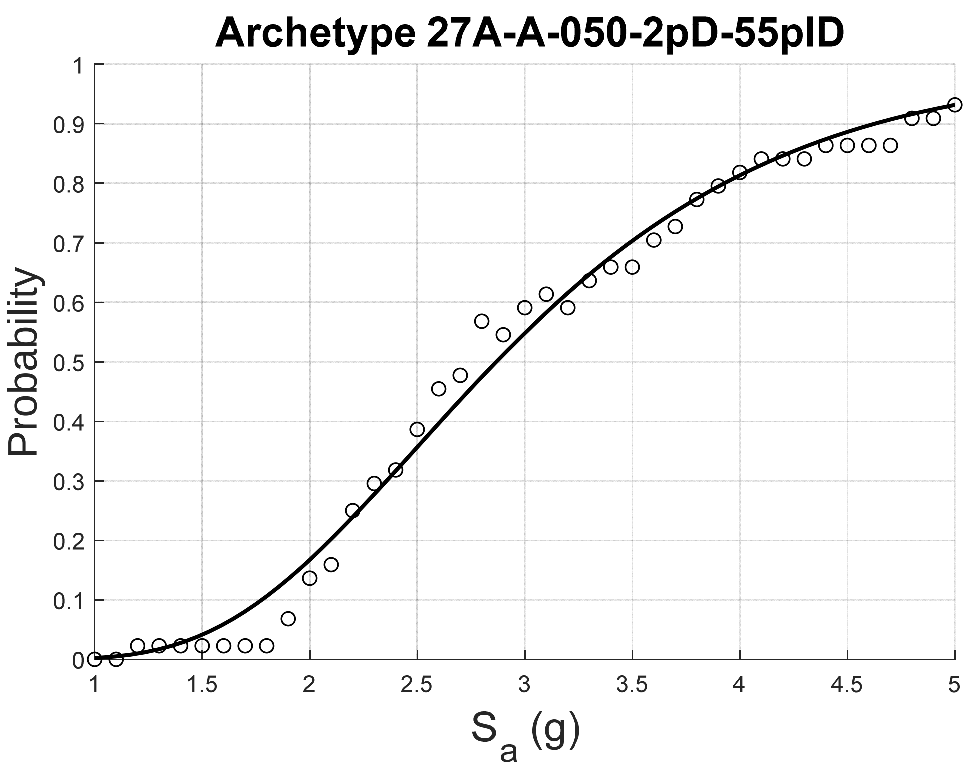


Figure G. 146. Archetype 27 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

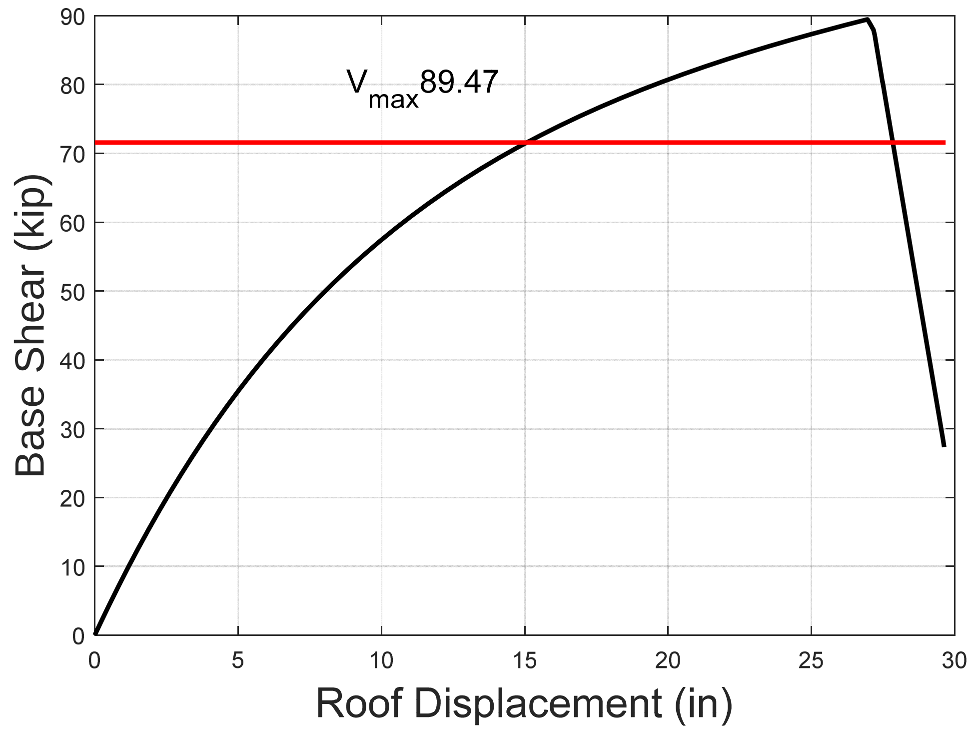


Figure G. 147. Archetype 28, R=4, static pushover

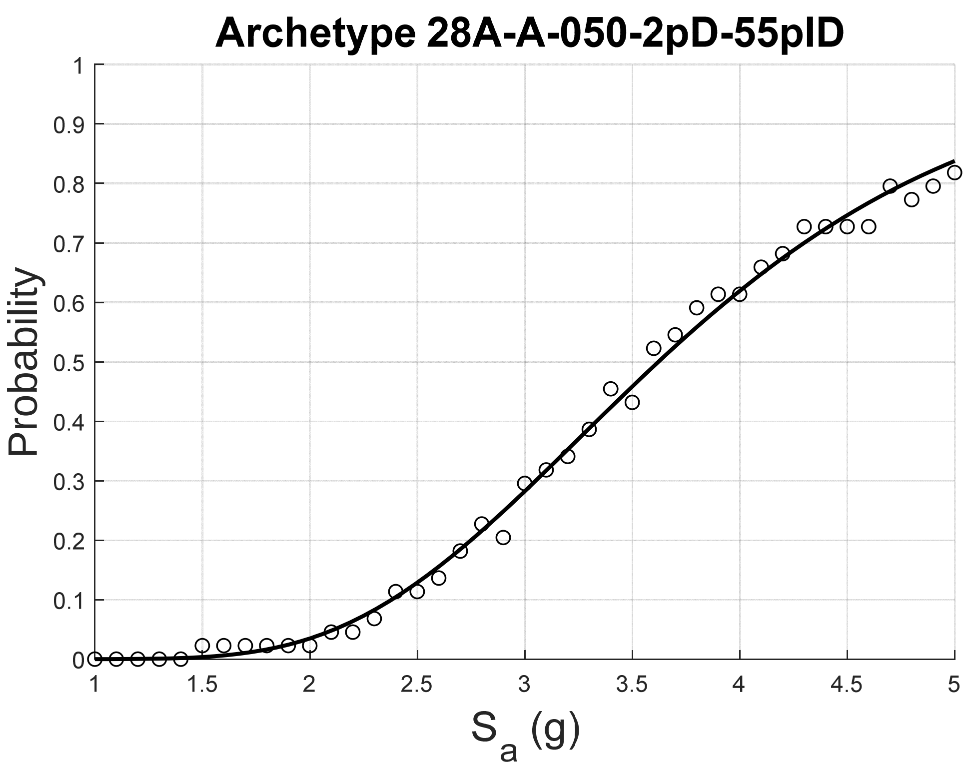


Figure G. 148. Archetype 28 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

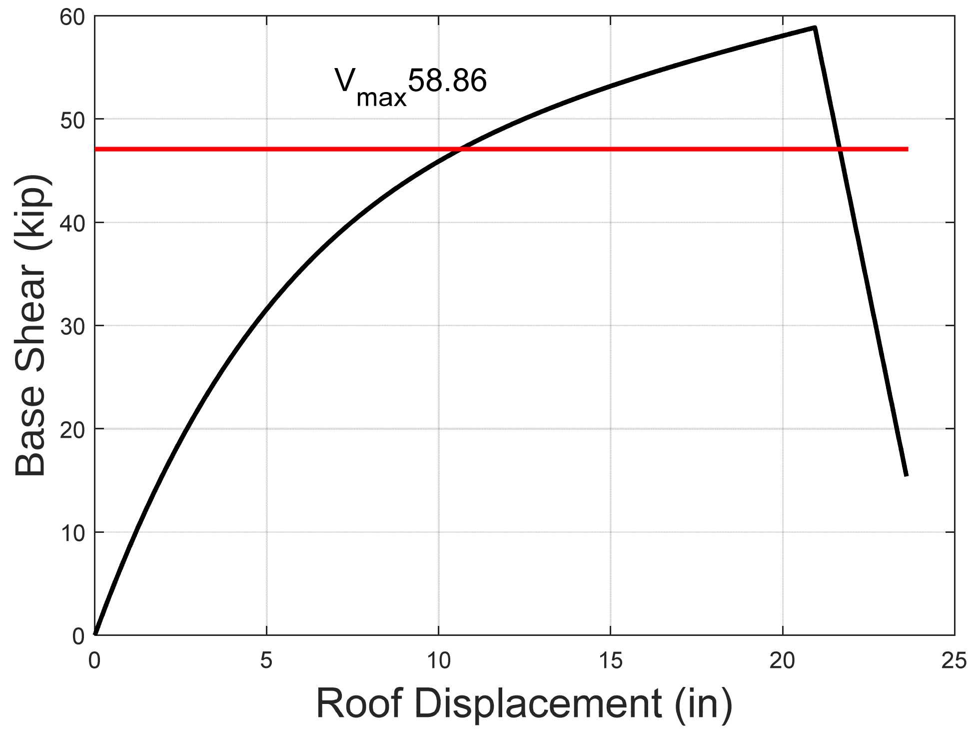


Figure G. 149. Archetype 29, R=4, static pushover

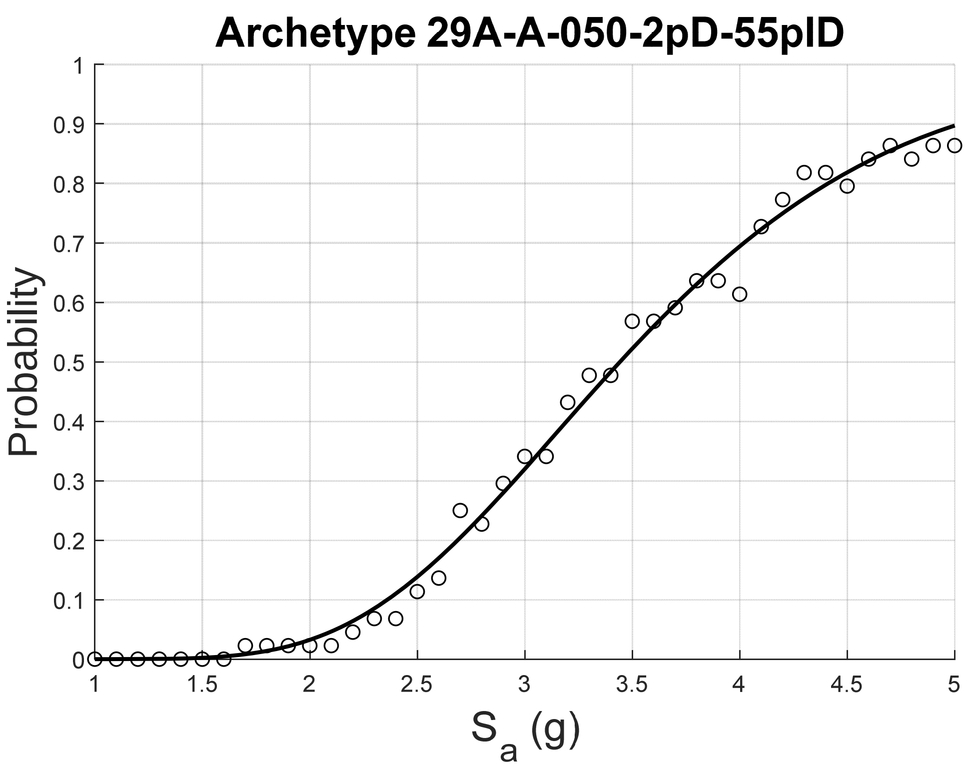


Figure G. 150. Archetype 29 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

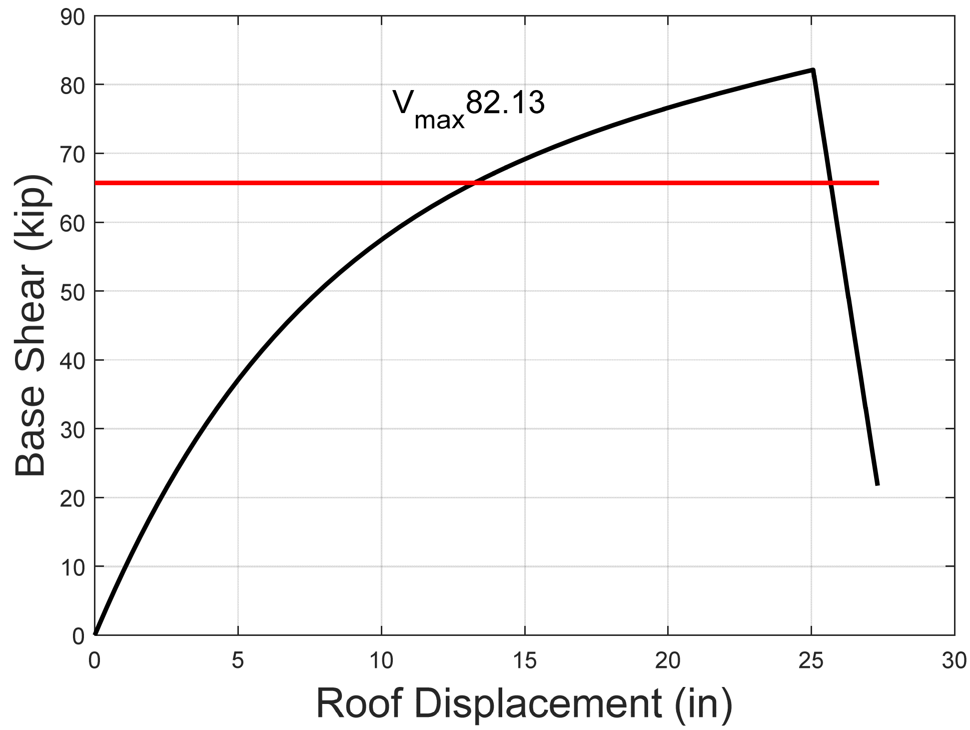


Figure G. 151. Archetype 30, R=4, static pushover

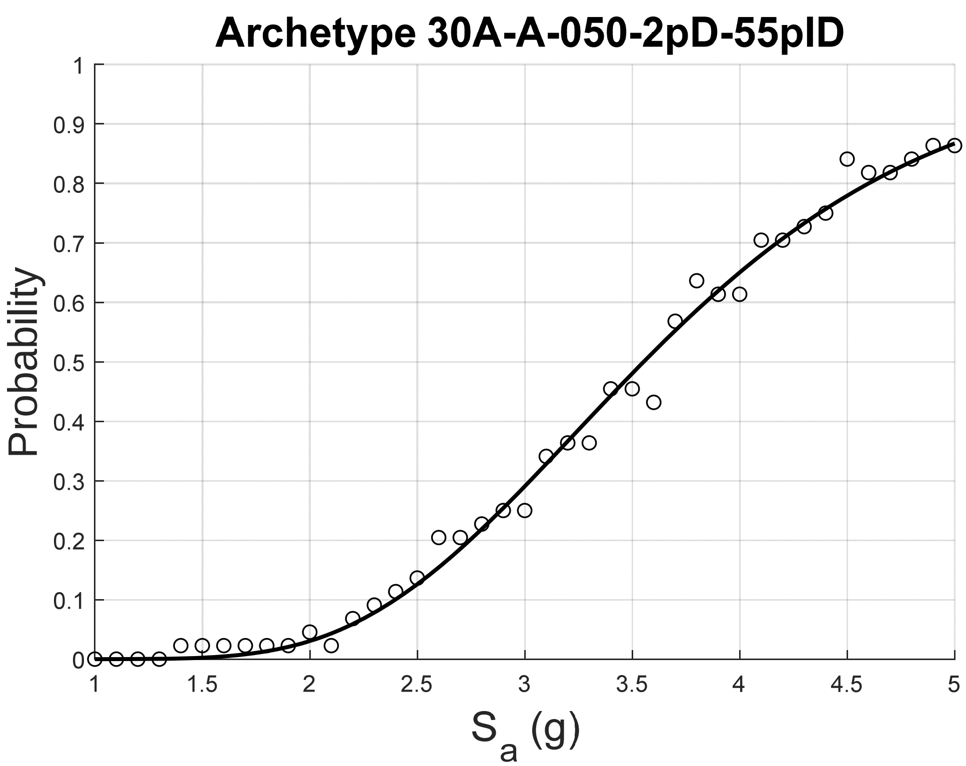


Figure G. 152. Archetype 30 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

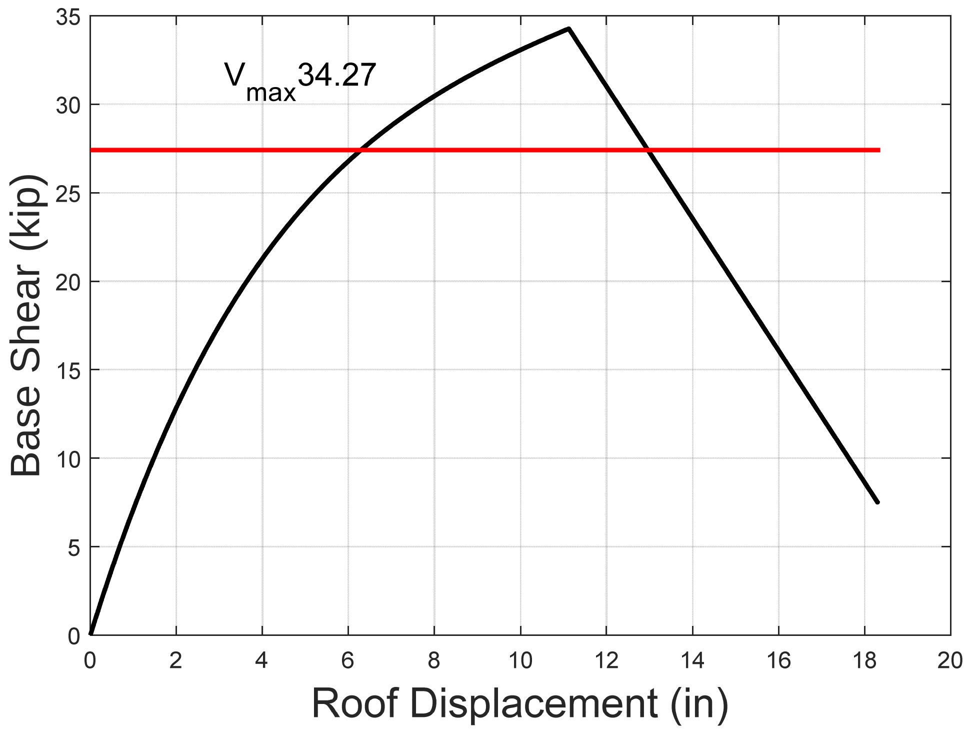


Figure G. 153. Archetype 37, R=4, static pushover

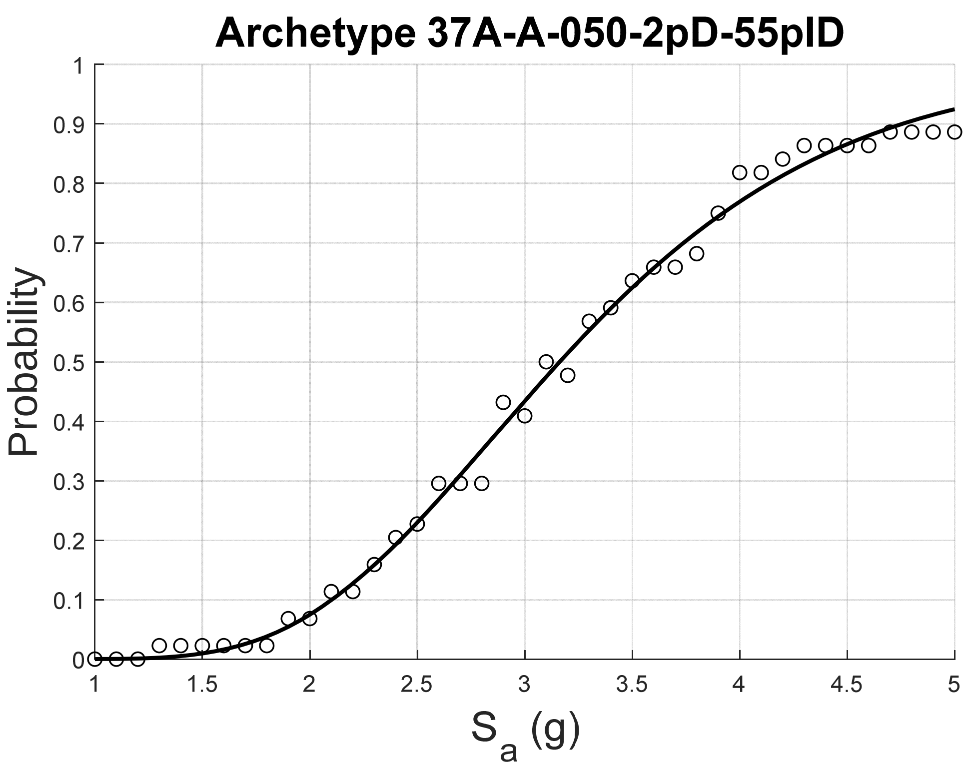


Figure G. 154. Archetype 37 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

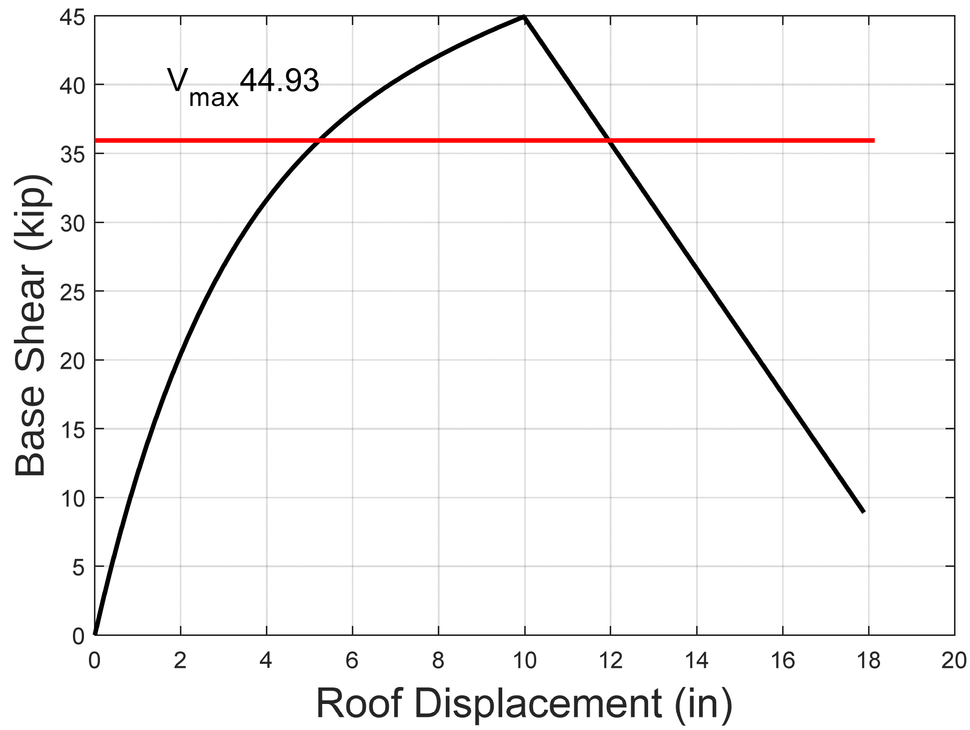


Figure G. 155. Archetype 38, R=4, static pushover

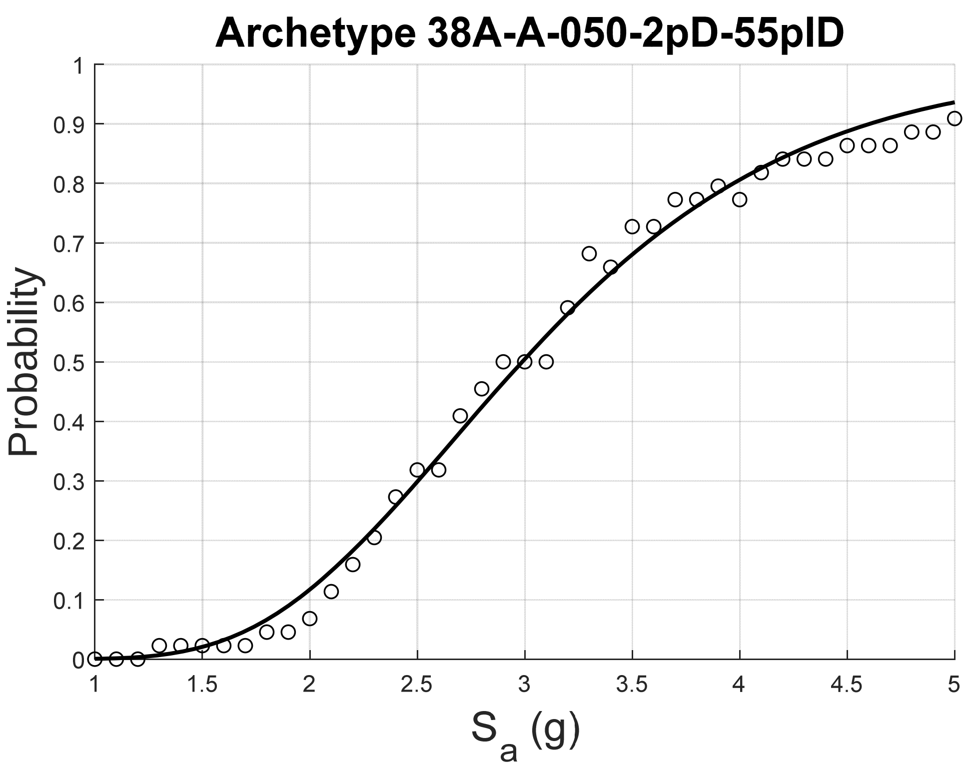


Figure G. 156. Archetype 38 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

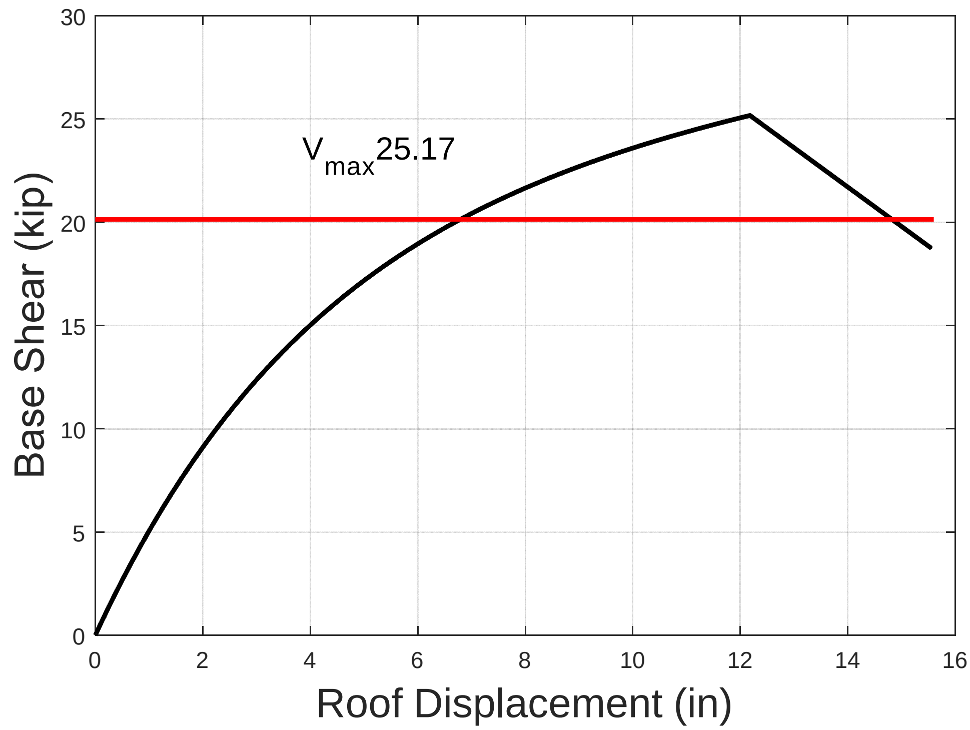


Figure G. 157. Archetype 39, R=4, static pushover

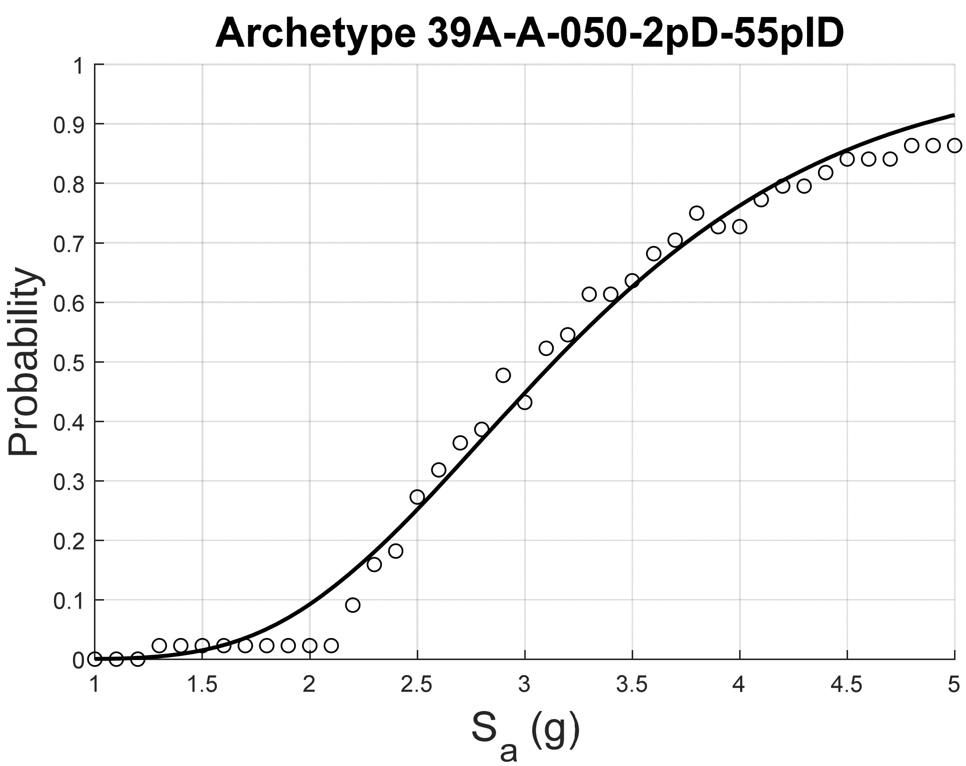


Figure G. 158. Archetype 39 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

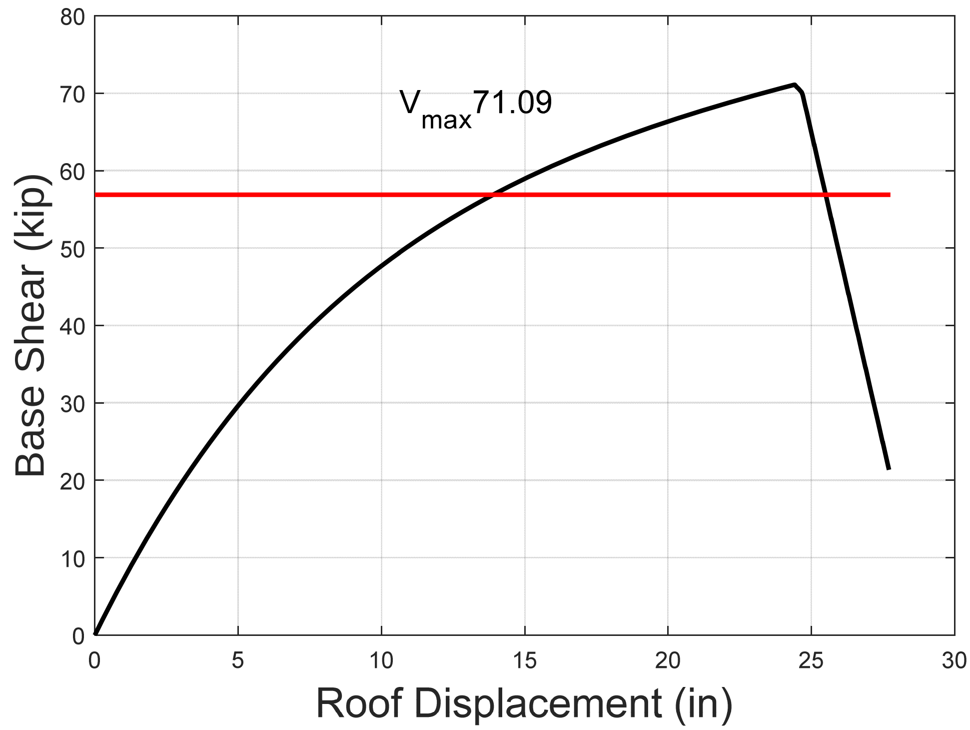


Figure G. 159. Archetype 40, R=4, static pushover

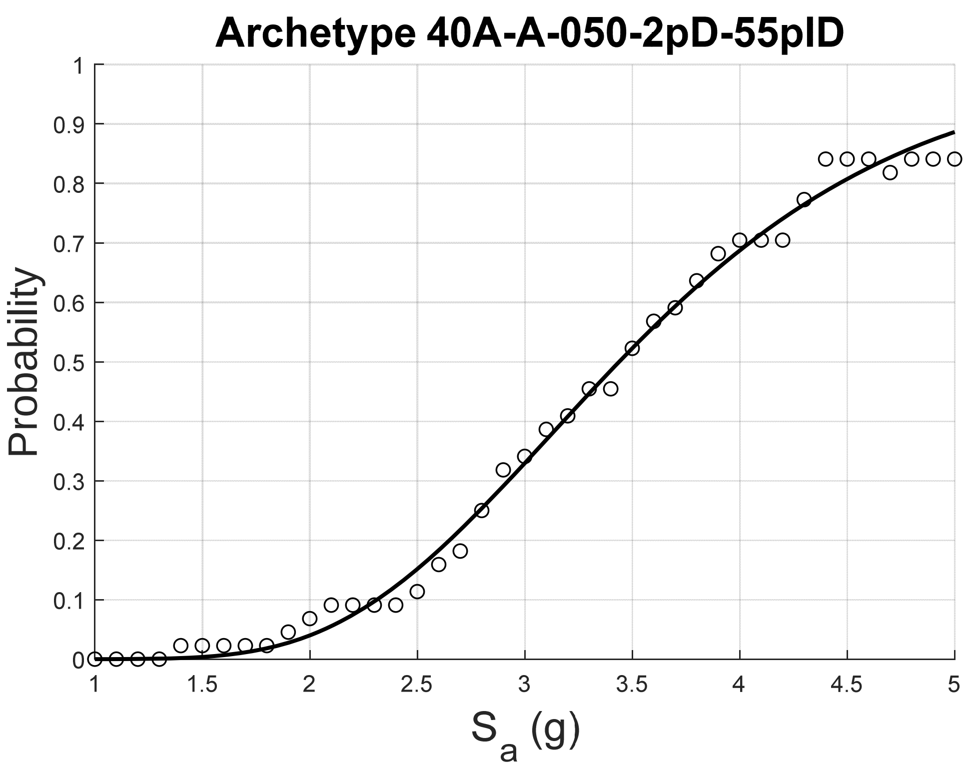


Figure G. 160. Archetype 40 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

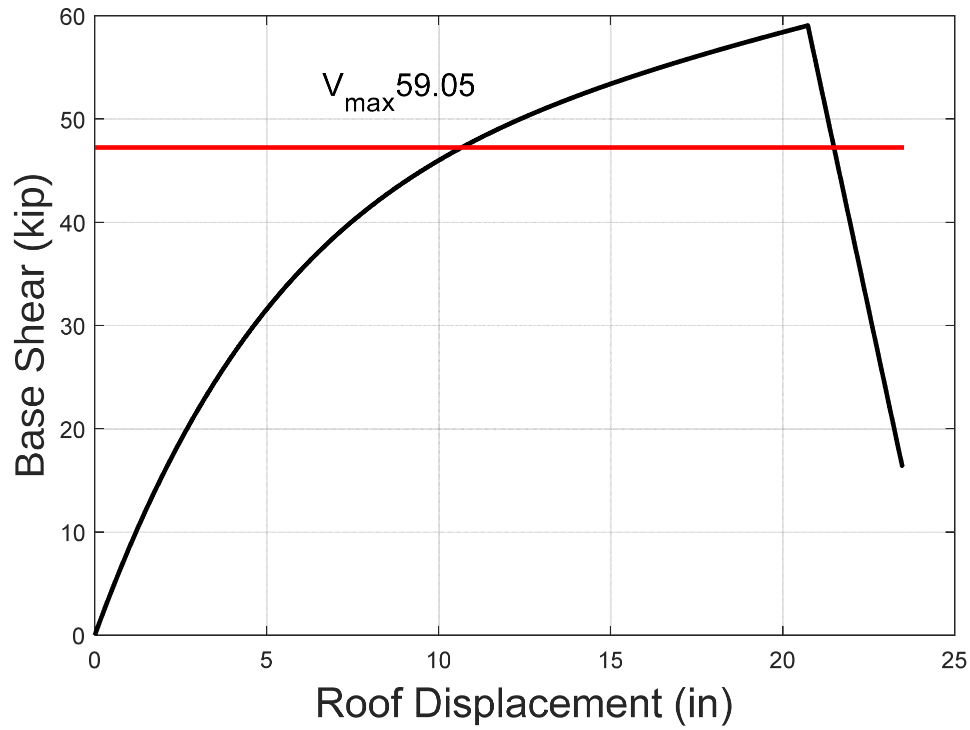


Figure G. 161. Archetype 41, R=4, static pushover

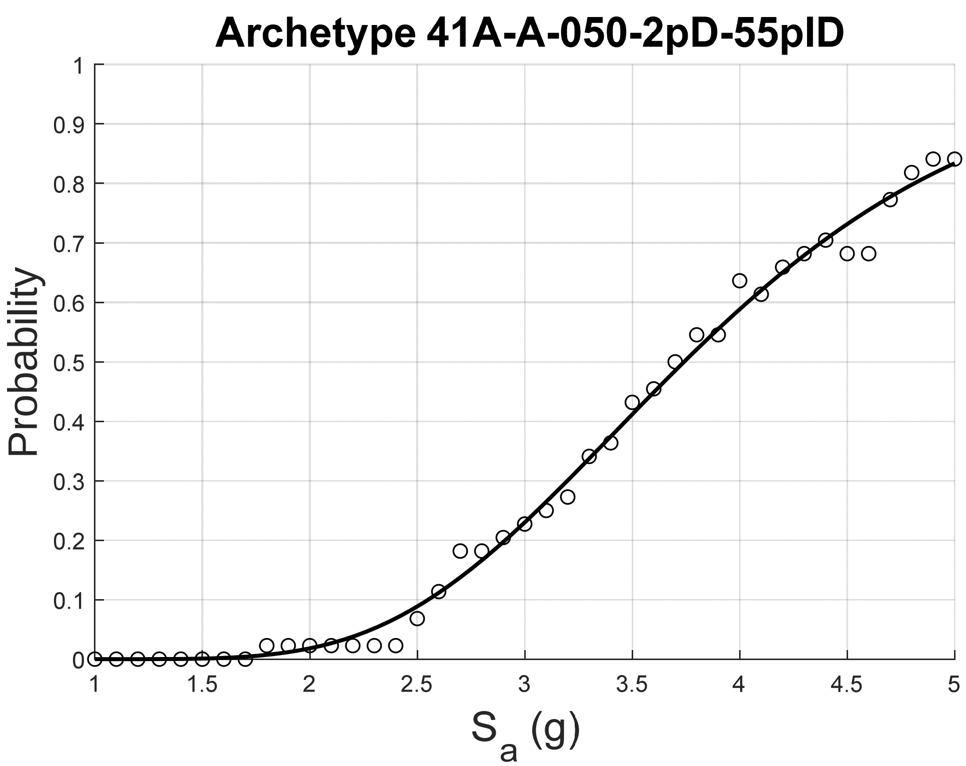


Figure G. 162. Archetype 41 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

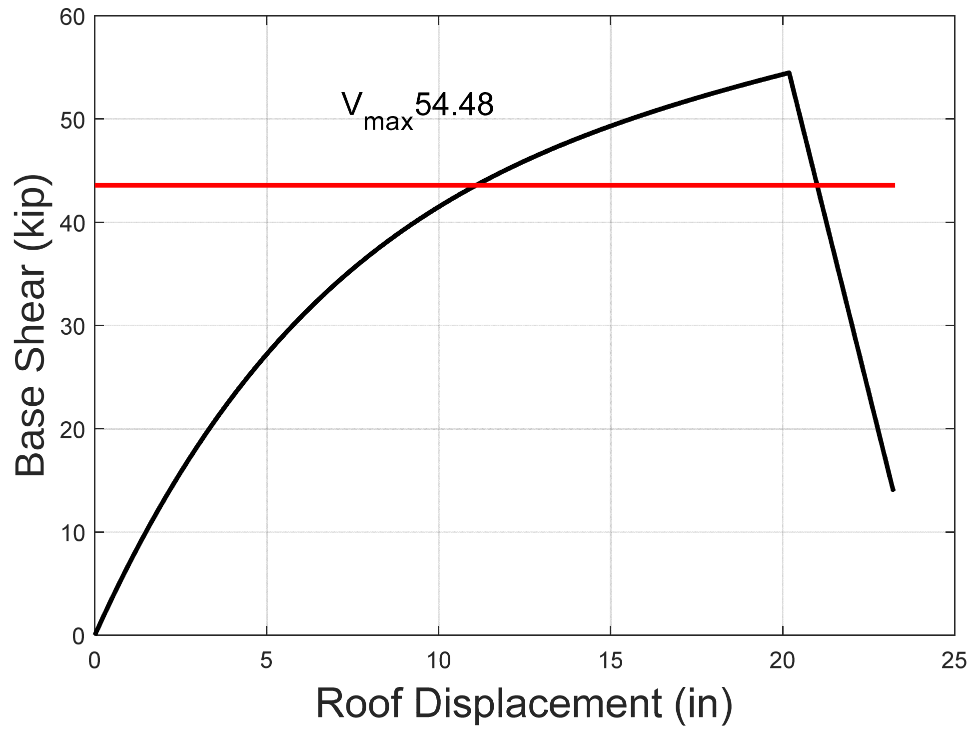


Figure G. 163. Archetype 42, R=4, static pushover

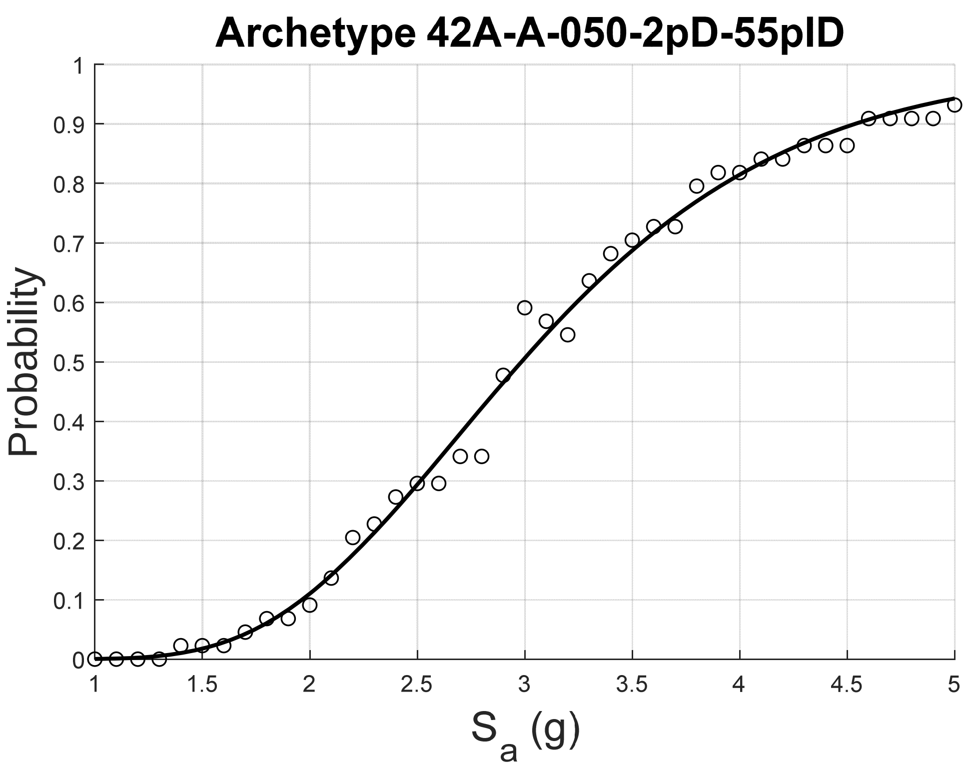


Figure G. 164. Archetype 42 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

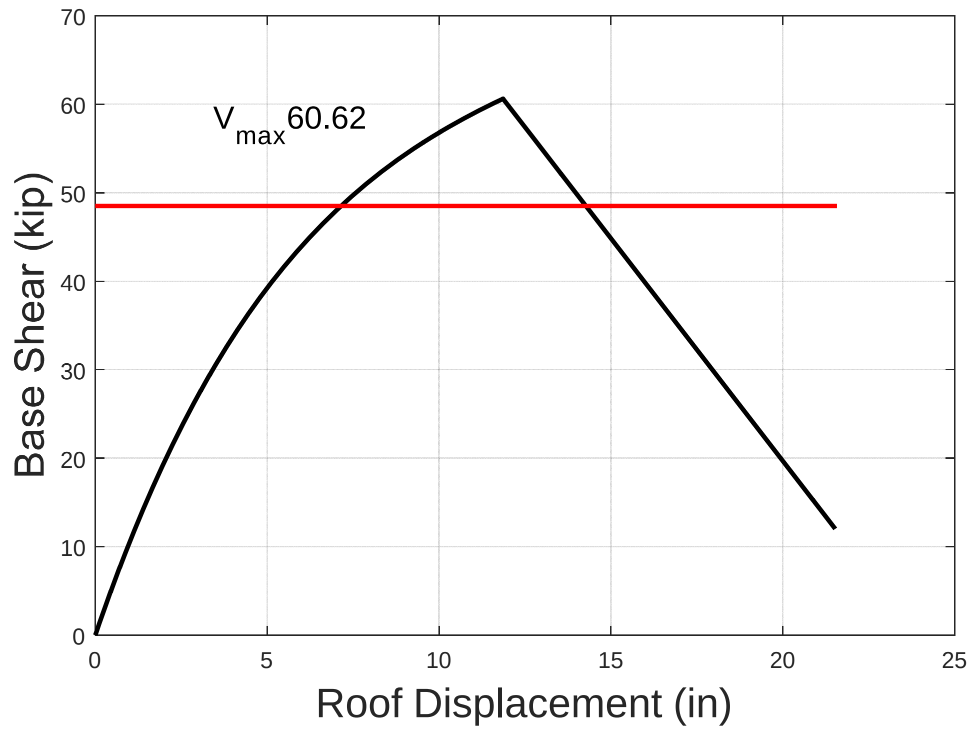


Figure G. 165. Archetype 97, R=4, static pushover

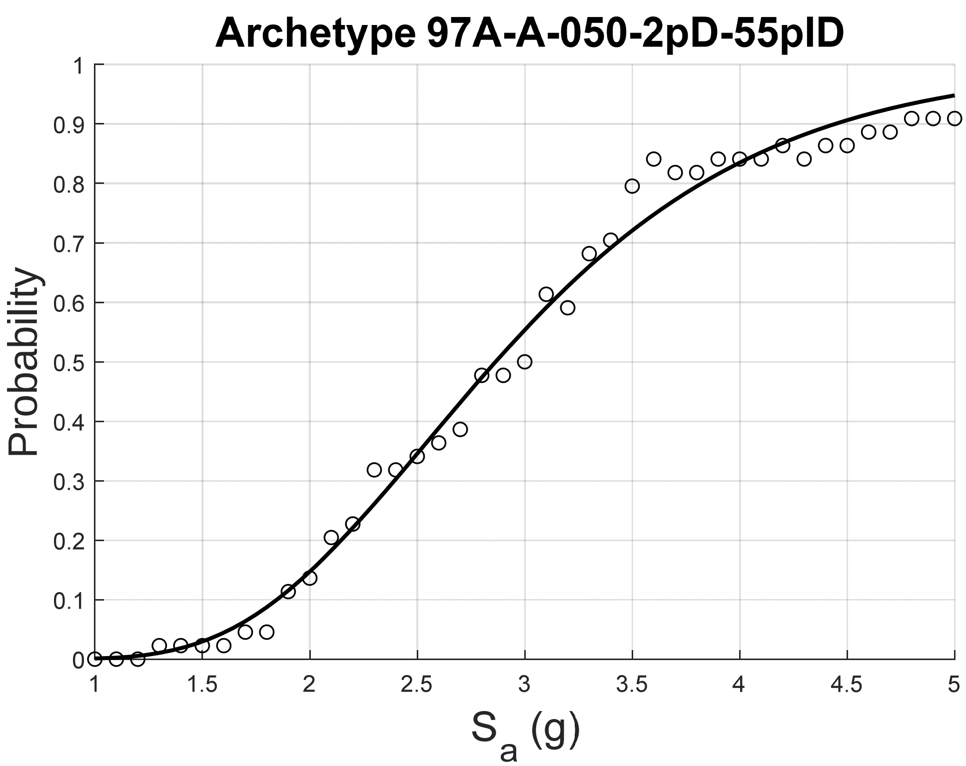


Figure G. 166. Archetype 97 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

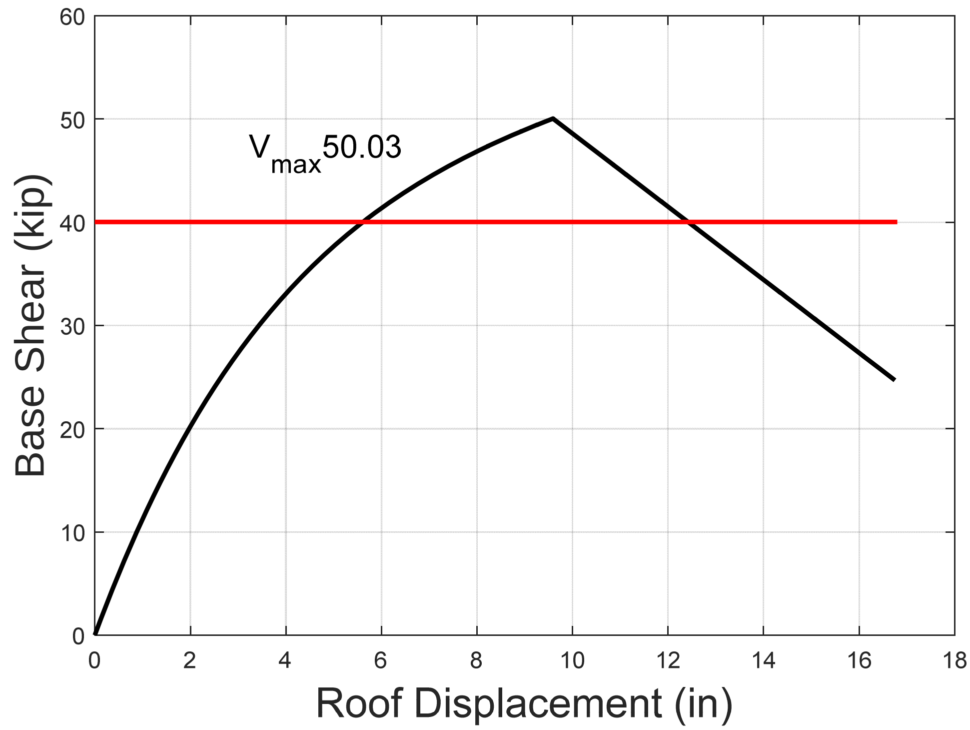


Figure G. 167. Archetype 98, R=4, static pushover

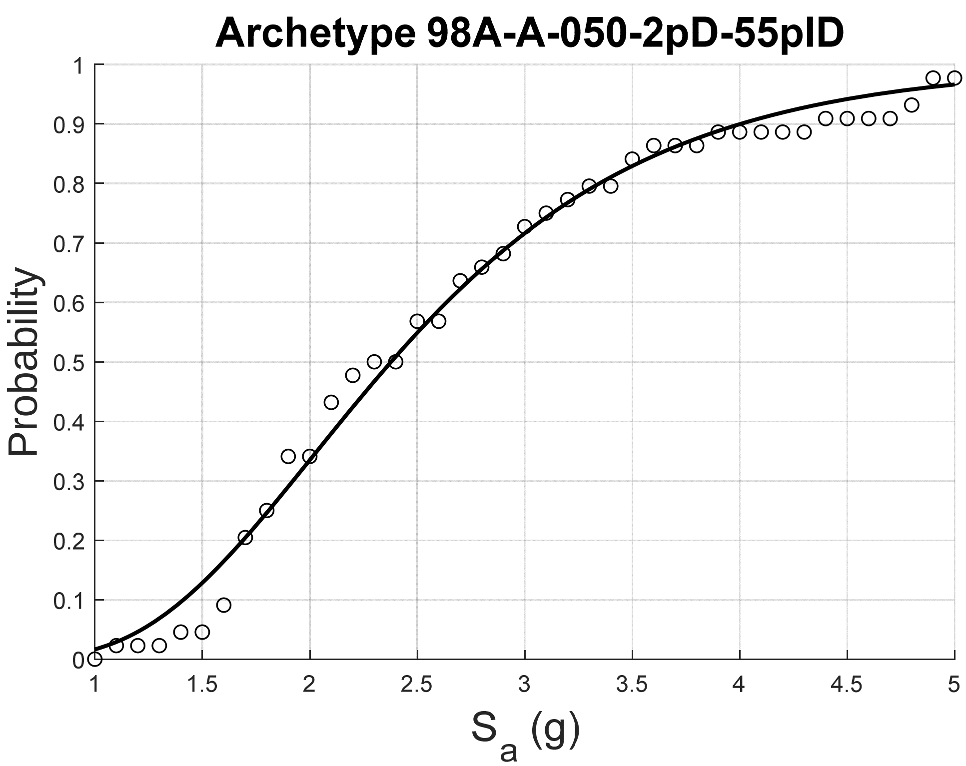


Figure G. 168. Archetype 98 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

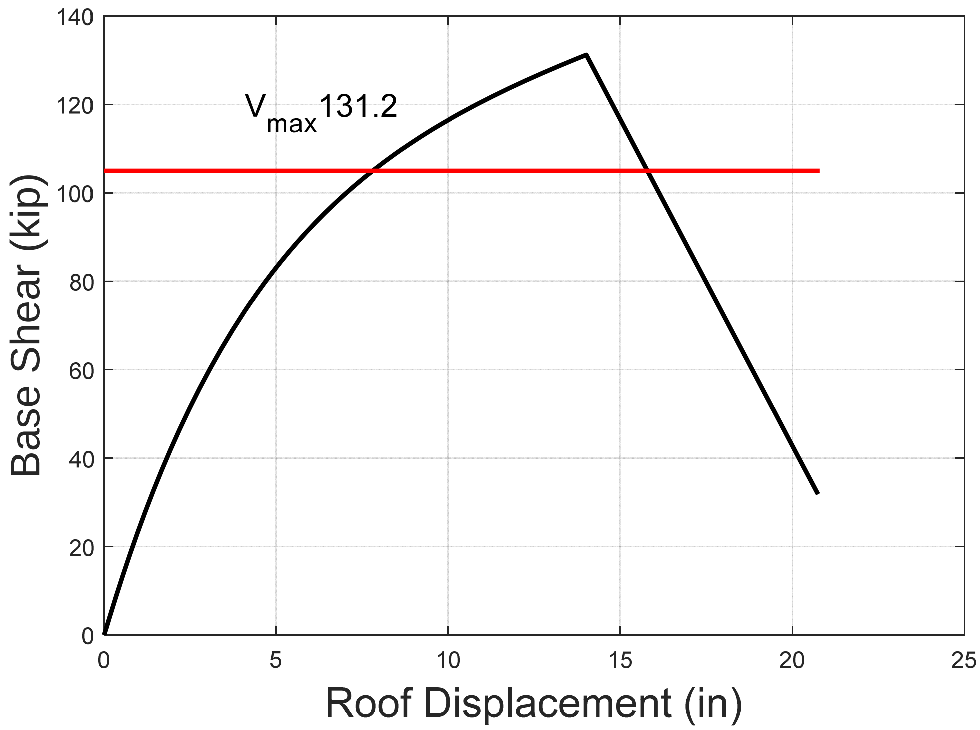


Figure G. 169. Archetype 99, R=4, static pushover

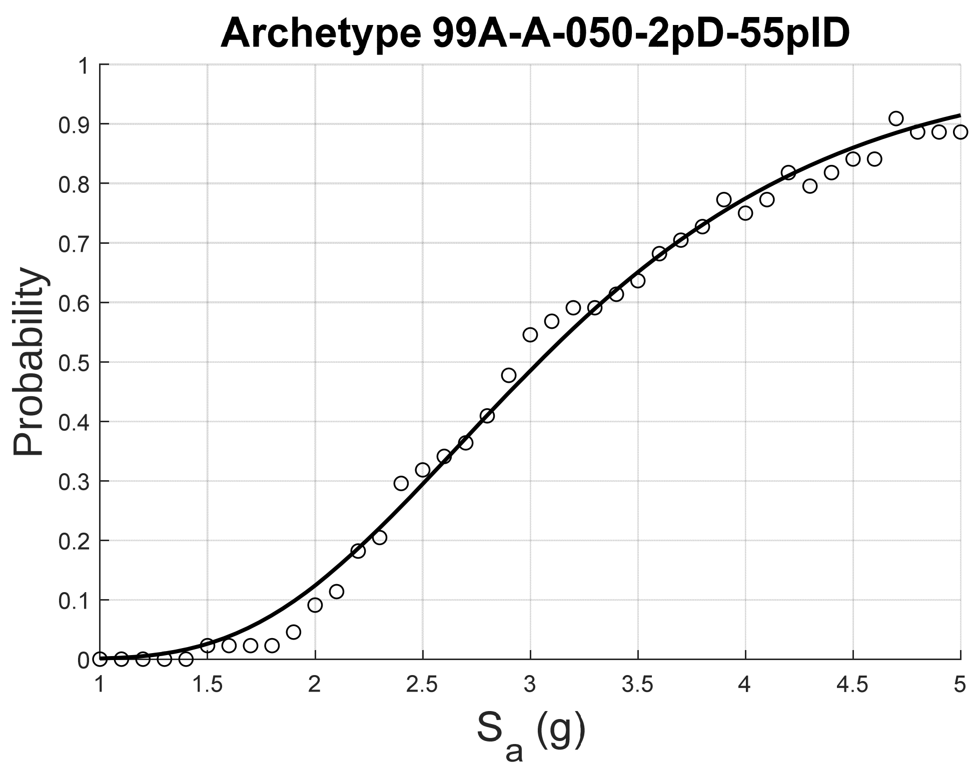


Figure G. 170. Archetype 99 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

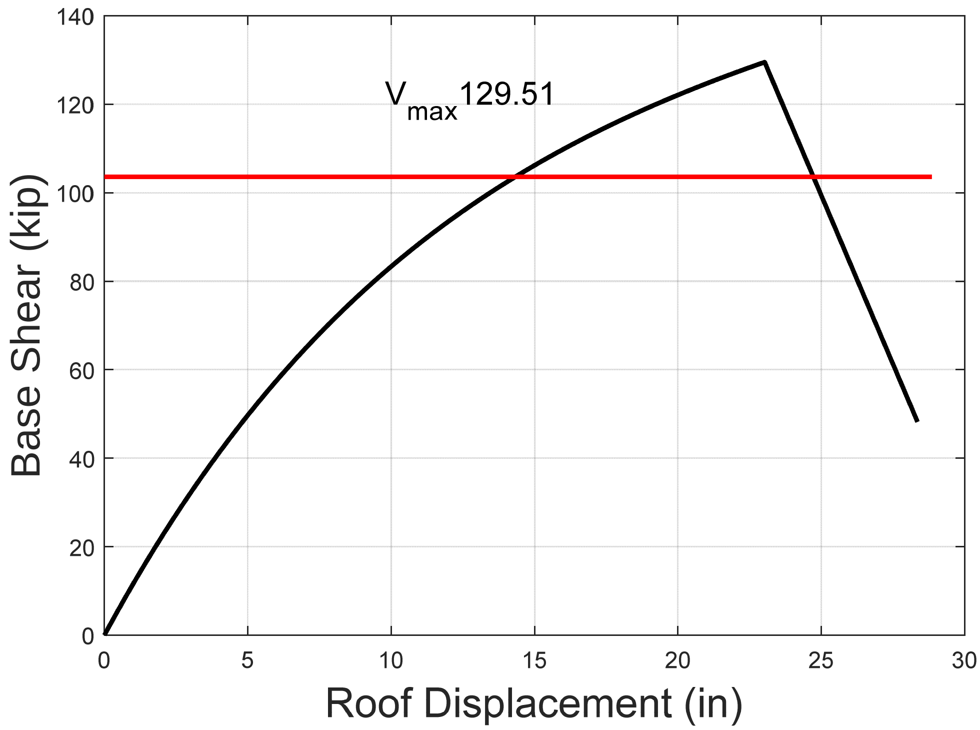


Figure G. 171. Archetype 100, R=4, static pushover

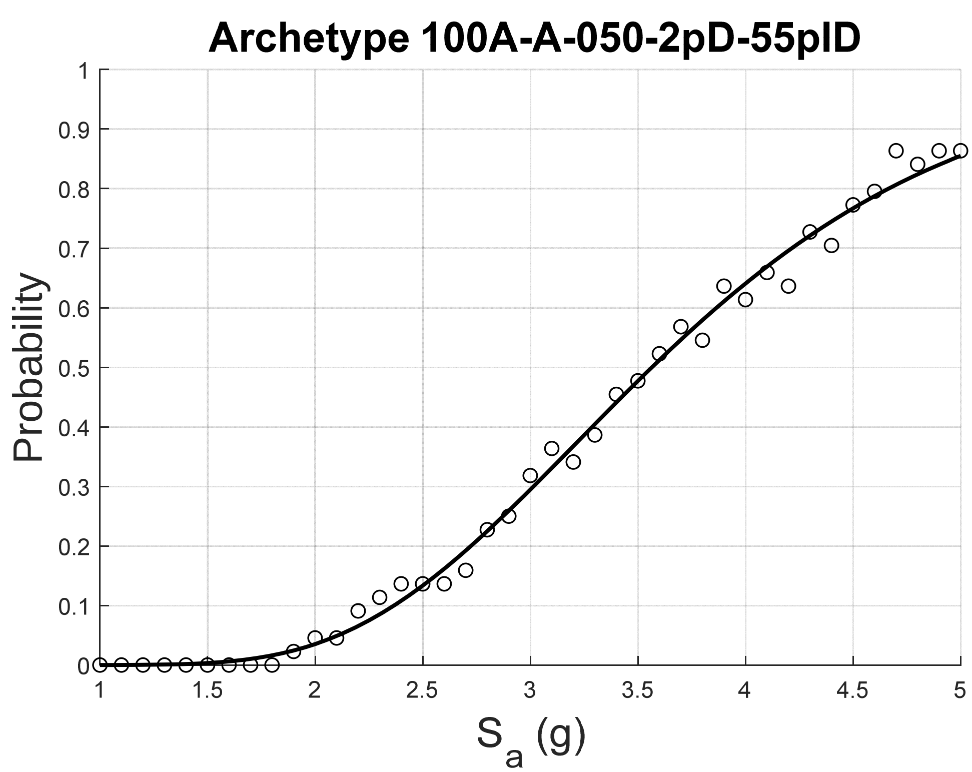


Figure G. 172. Archetype 100 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

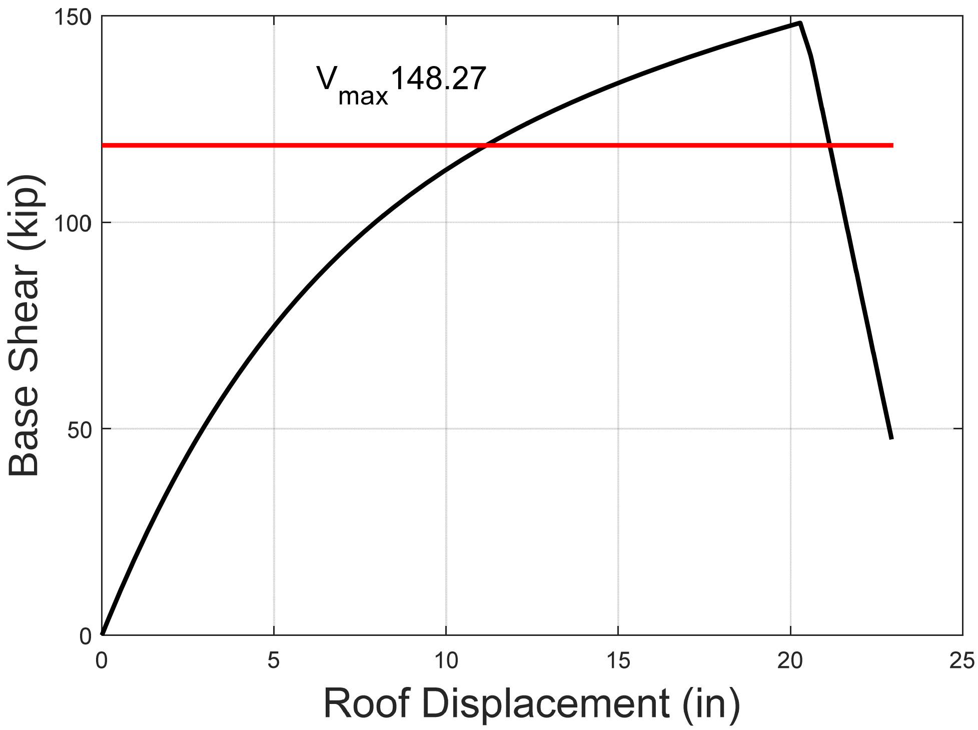


Figure G. 173. Archetype 101, R=4, static pushover

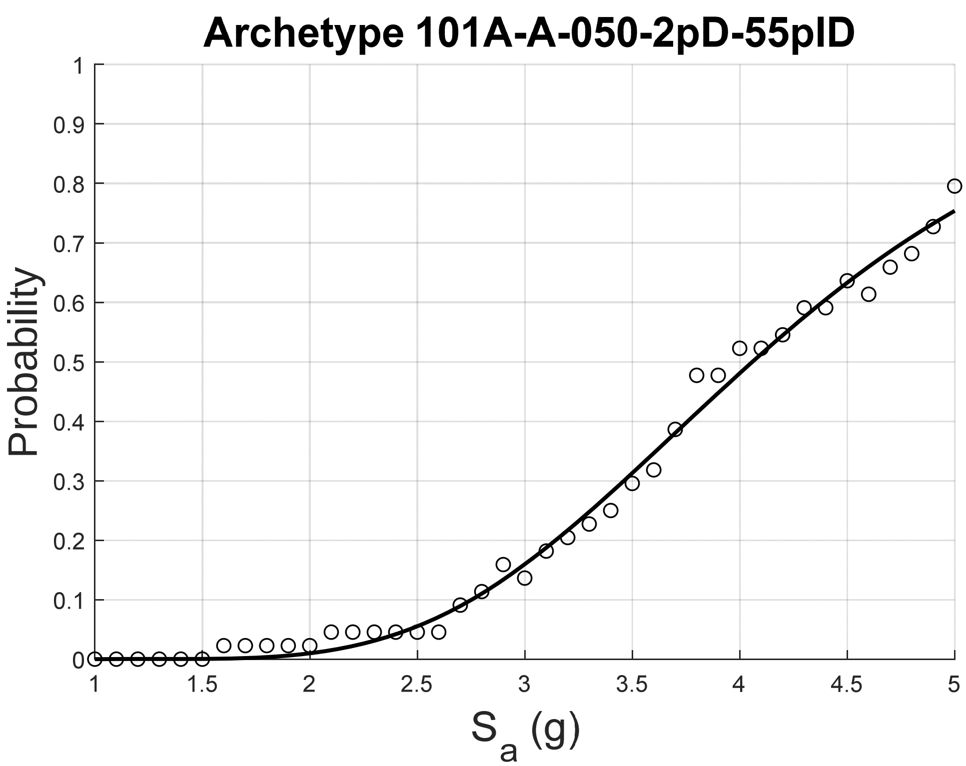


Figure G. 174. Archetype 101 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

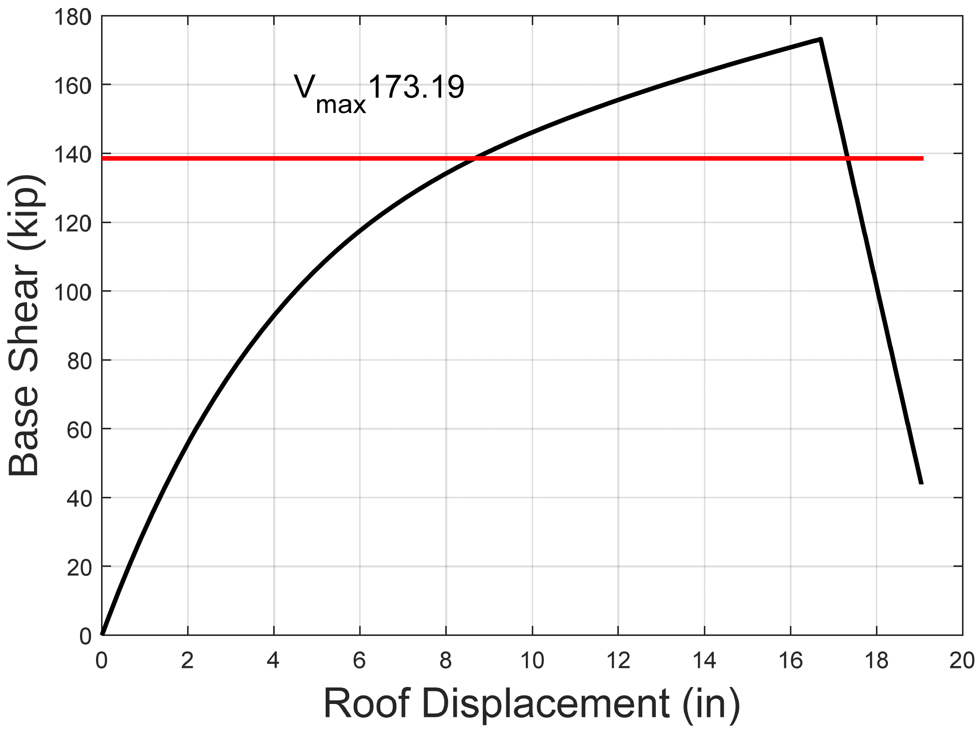


Figure G. 175. Archetype 102, R=4, static pushover

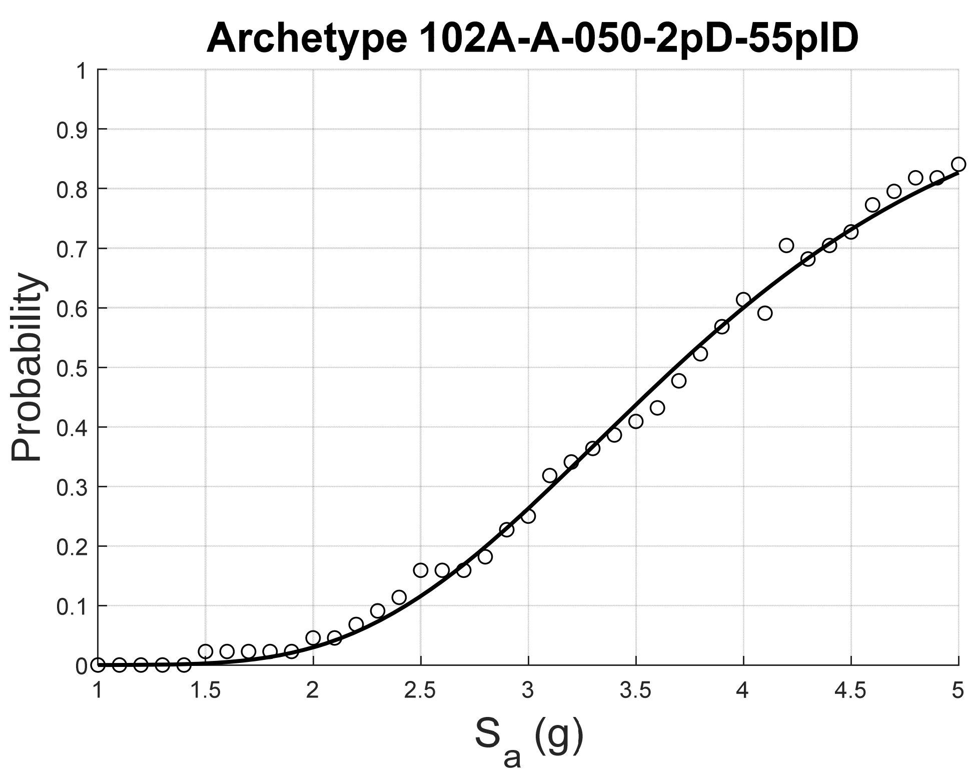


Figure G. 176. Archetype 102 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

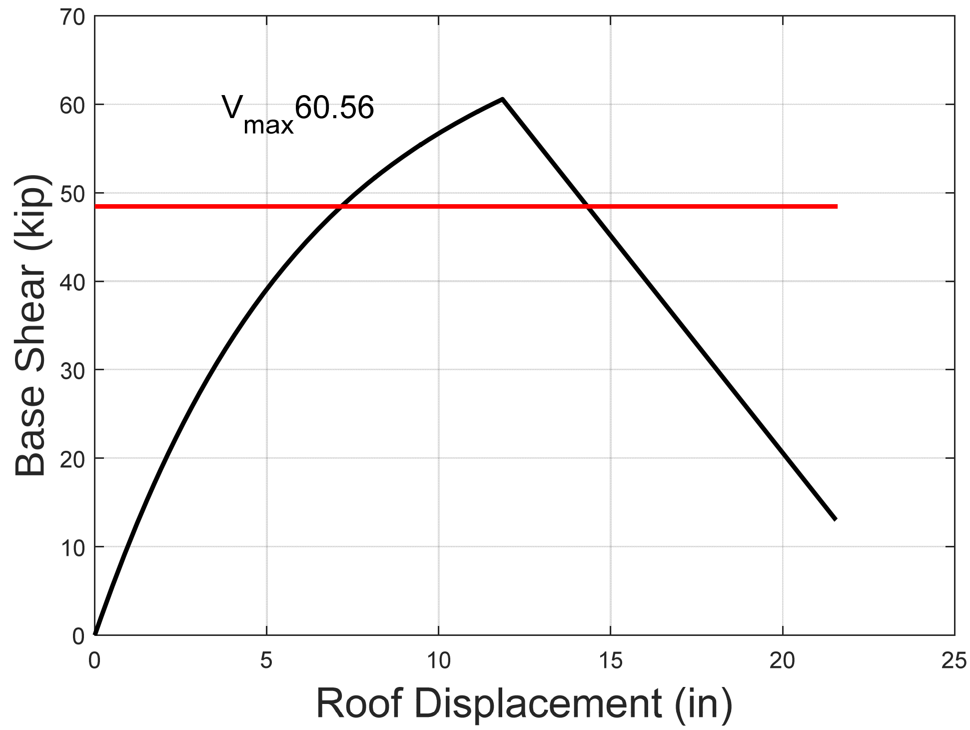


Figure G. 177. Archetype 109, R=4, static pushover

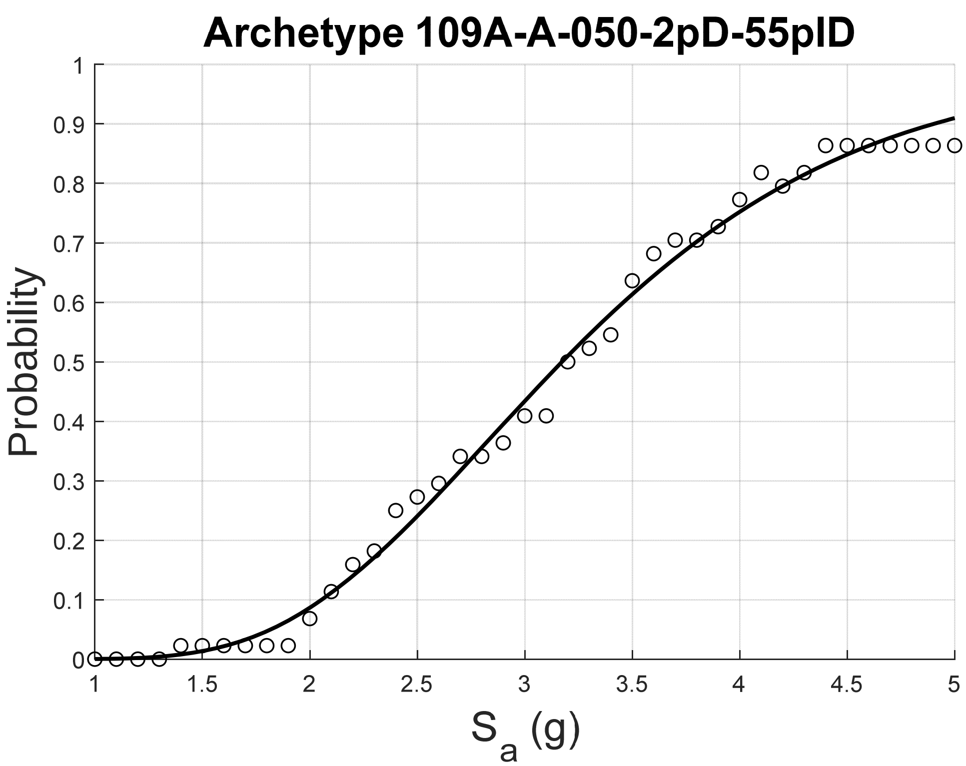


Figure G. 178. Archetype 109 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

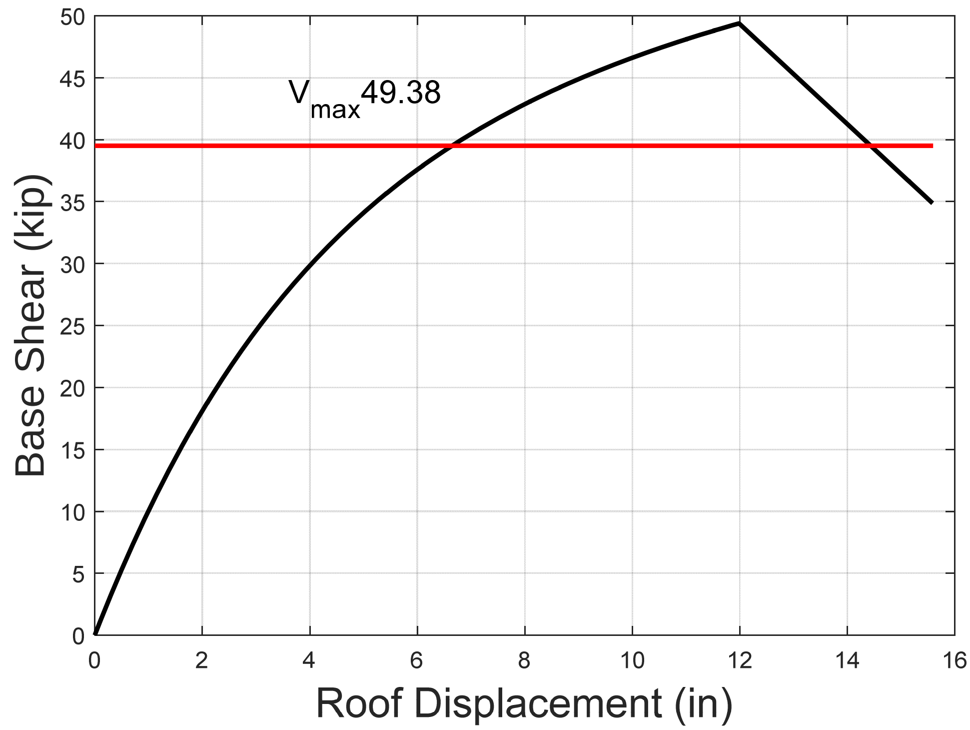


Figure G. 179. Archetype 110, R=4, static pushover

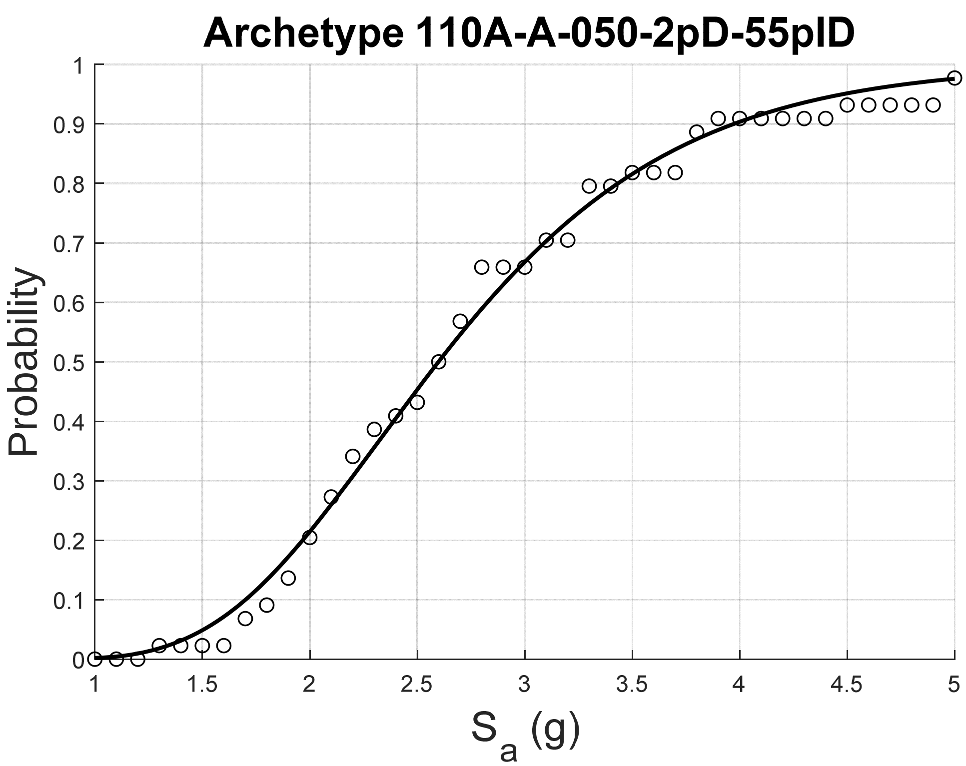


Figure G. 180. Archetype 110 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

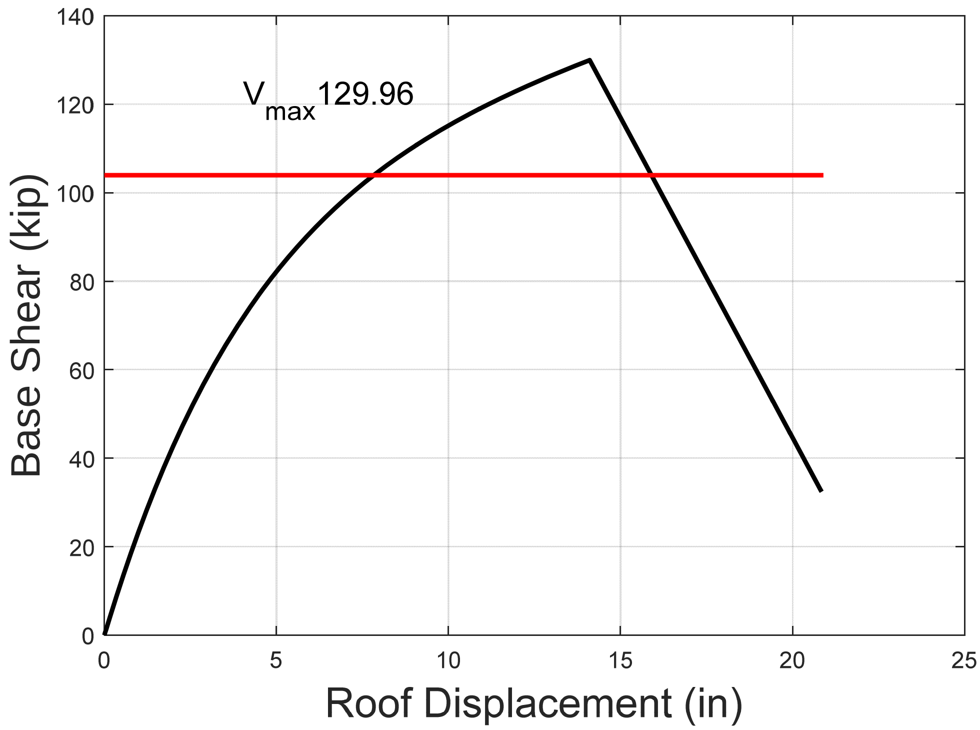


Figure G. 181. Archetype 111, R=4, static pushover

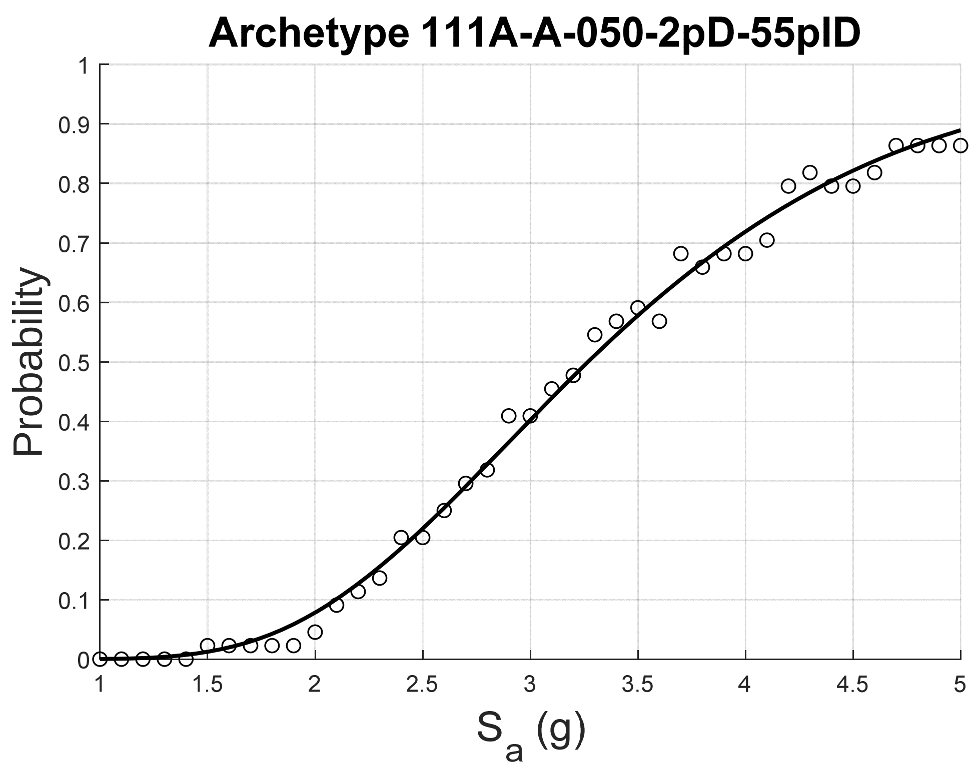


Figure G. 182. Archetype 111 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit

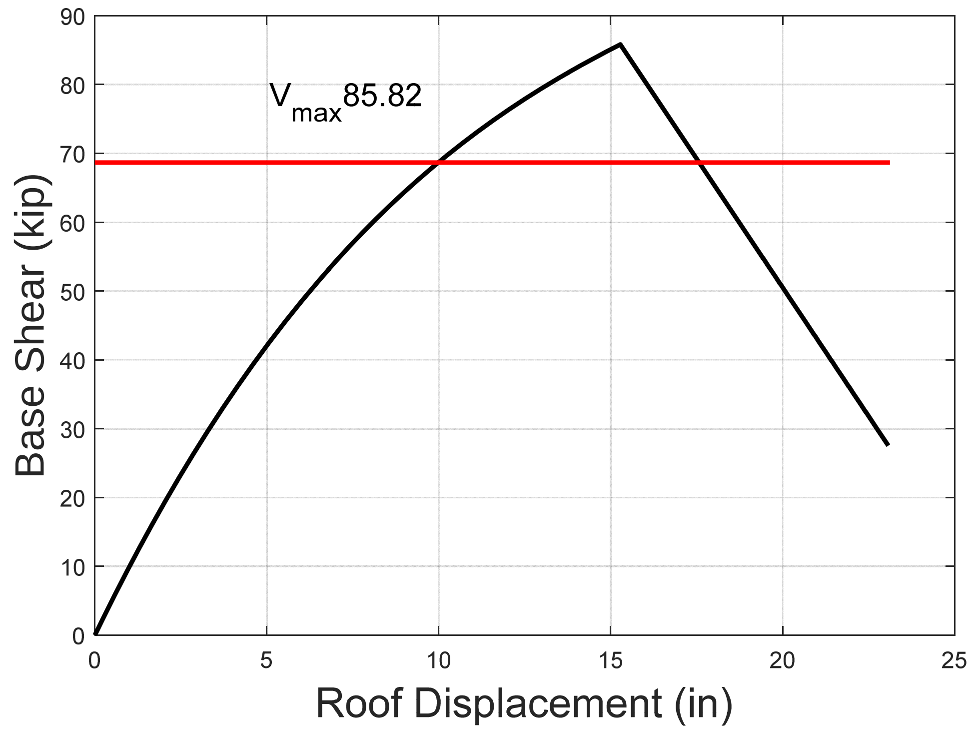


Figure G. 183. Archetype 112, R=4, static pushover

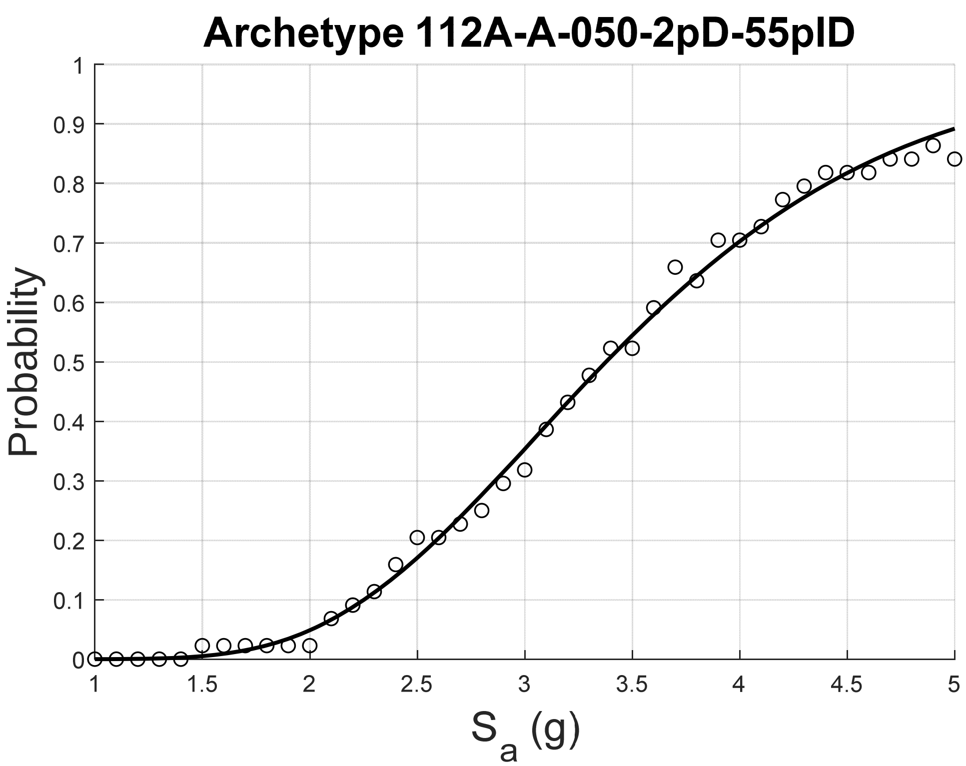


Figure G. 184. Archetype 112 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

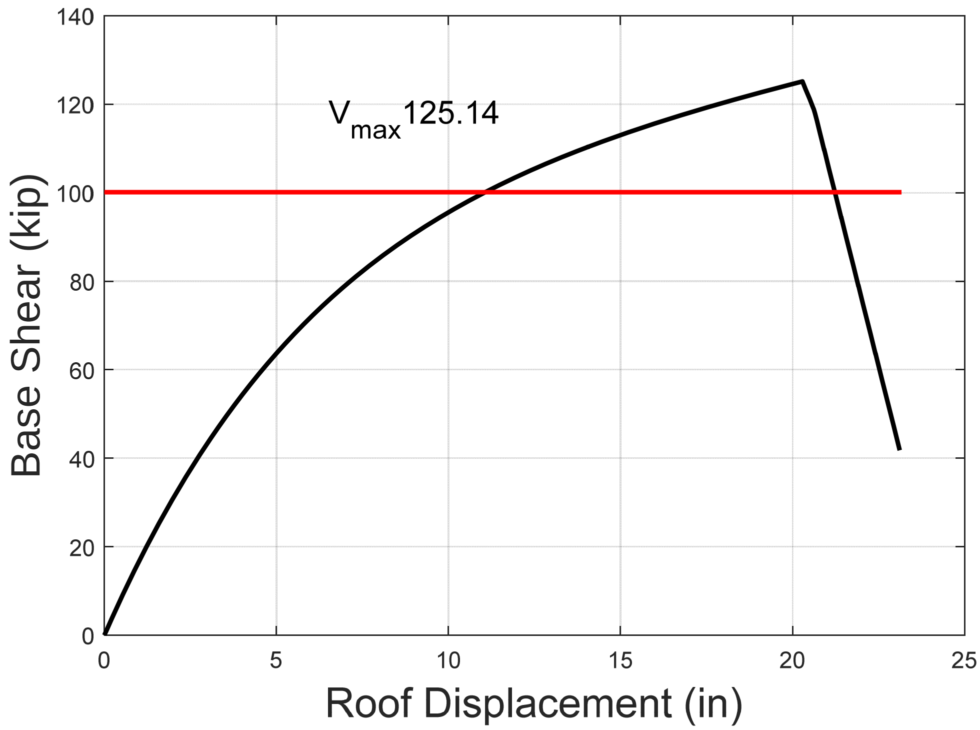


Figure G. 185. Archetype 113, R=4, static pushover

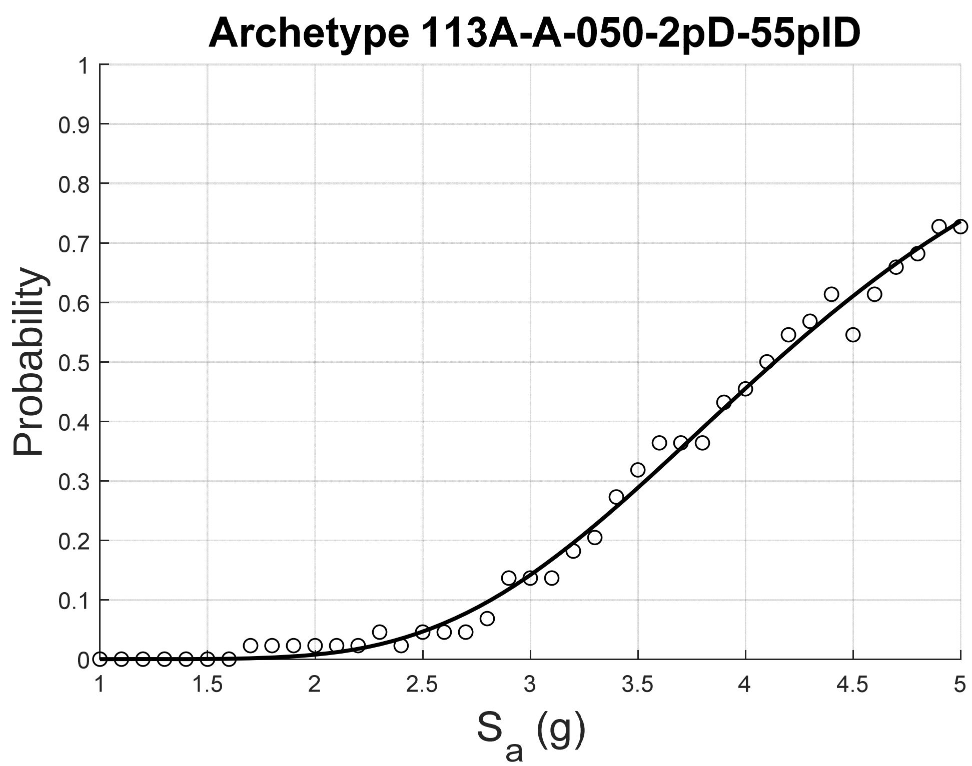


Figure G. 186. Archetype 113 collapse fragility R=4, 2% damping, 5.5% interstory drift limit

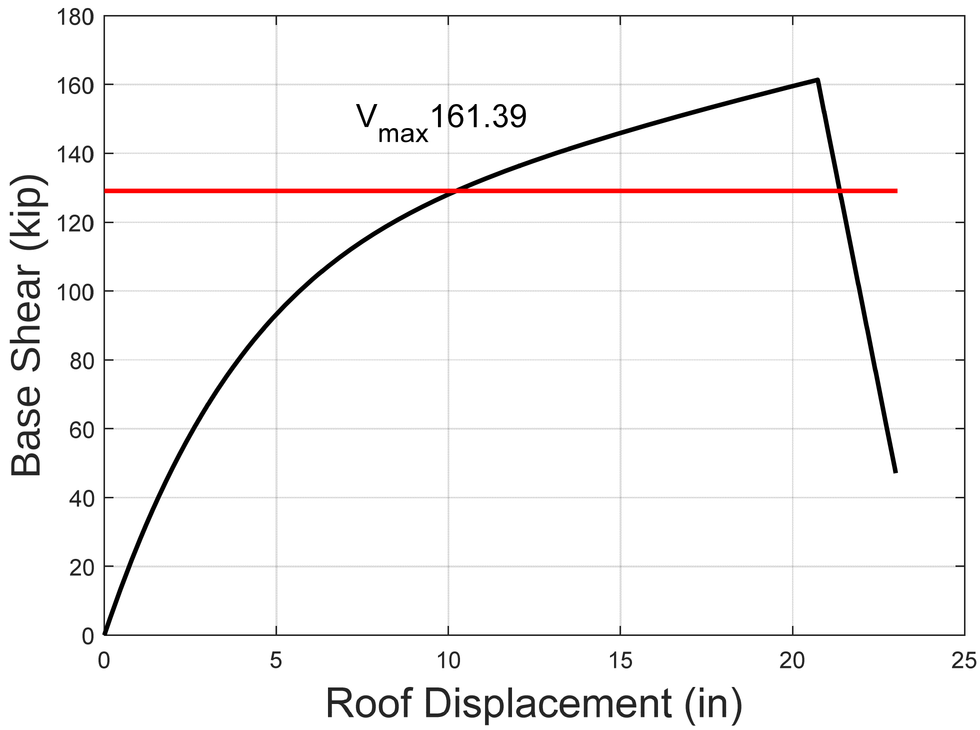


Figure G. 187. Archetype 114, R=4, static pushover

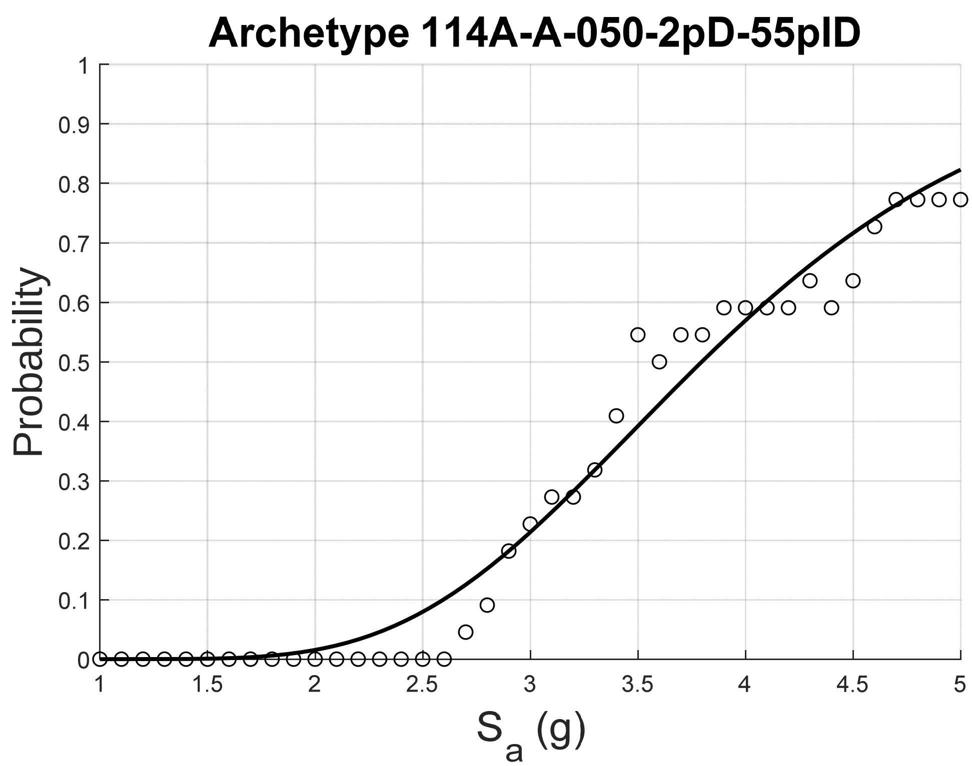


Figure G. 188. Archetype 114 collapse fragility
R=4, 2% damping, 5.5% interstory drift limit