**A Study of the Seismic Performance of Early
Multi-Story Steel Frame Structures with Unreinforced
Masonry Infill**

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A STUDY OF THE SEISMIC PERFORMANCE OF EARLY MULTI-STORY STEEL
FRAME STRUCTURES WITH UNREINFORCED MASONRY INFILL

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ABSTRACT

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Steel frame construction with unreinforced masonry infill walls is a common system found in high-rise structures built in the late nineteenth and early twentieth centuries. Recorded performance of this dual system during seismic events shows that the structures are able to resist a high level of lateral loads without collapse, primarily because a majority of damage is confined to the infill walls instead of the gravity carrying frame. To better understand expected performance of this structural system in different seismic risk regions, a prototypical building was analyzed using modal and nonlinear static procedures based on currently accepted evaluation guidelines. Nonlinear results from the computer model were compared with calculated target displacements for seventeen cities likely to have steel frame construction with unreinforced masonry infill in order to determine expected damage levels at varying levels of seismic risk. It was concluded that the structural system studied could experience damage in all seismic risk regions, including post-yield damage of the structure, although in low risk regions that damage is confined entirely to the infill walls. Practicing structural engineers should be aware that in all seismic risk zones existing steel frame buildings with unreinforced masonry infill, while able to resist a high magnitude of displacement without complete structural failure, will require additional lateral support under currently accepted rehabilitation guidelines.

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List of Nomenclature

a	equivalent width of infill strut
C_0	modification factor to relate spectral displacement of an equivalent single-degree of freedom system to the roof displacement of the building multi-degree of freedom system
C_1	modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
C_2	modification factor to represent the effects of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response
h_{col}	column height between centerlines of beams (in)
h_{inf}	height of infill panel (in)
E_{fe}	expected modulus of elasticity of frame material (ksi)
E_{me}	expected modulus of elasticity of infill material (ksi)
g	acceleration of gravity
I_{col}	moment of inertia of column (in ⁴)
K_e	effective lateral stiffness of the building in the direction under consideration (k/in)
K_i	elastic lateral stiffness of the building in the direction under consideration (k/in)
L_{inf}	length of infill panel (in)
r_{inf}	diagonal length of infill panel (in)
S_a	spectral response acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration
S_1	spectral response acceleration parameter at a one-second period
S_s	spectral response acceleration parameter at short periods
T_e	effective fundamental period of the building in the direction under consideration (seconds)

T_i	elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis (seconds)
t_{inf}	thickness of infill panel and equivalent strut (in)
δ_t	target displacement
θ	angle whose tangent is the infill height-to-length aspect ratio (radians)
λ_I	coefficient used to determine equivalent width of infill strut

1.0 STATEMENT OF THE PROBLEM

The casual observers on a street in a metropolis like San Francisco or New York may not realize that the late nineteenth- and early twentieth-century structures surrounding them are actually steel frame structures. Because buildings of this early era in steel construction are generally clad in masonry, they provide a sharp contrast to the austere steel frames of modern design and may therefore be overlooked as forerunners of modern steel construction. However, the combination of steel and masonry infill wall construction used in early steel designs, such as the Mills Building shown in **Figure A** (below), created a structural system whose performance capabilities influenced high rise construction for most of twentieth century.

Figure A: The Mills Building (1891), San Francisco, CA
Source: Corbett 1979

By focusing on multi-story steel framed construction with unreinforced masonry (URM) infill walls constructed between the 1880s and 1930s in the United States, a better

appreciation of both this early era of steel construction and later advances in modern steel design will be developed. Studying the performance and design of early high-rise steel construction will also give insight to the capabilities of those structures and variation in expected performance issues for different seismic risk regions.

This two-part project consists of an in-depth literature review and the study of a prototypical building. The literature review focuses on the history, construction, materials, analysis, and performance of steel frame construction with URM infill walls, a dual structural system. The information gathered during the literature review and by meeting with practicing structural engineers helped to create a prototypical building to represent the structural system, allowing for the study of behavioral variation in different seismic risk zones. Additionally, a better understanding of the modeling techniques used with the structural system was gained by completing a computer model. Due to recent building code changes in seismic design, the modeling and study of a single structure in different risk regions gives a better understanding of how rehabilitation efforts in lower seismic risk regions of the United States has been affected.

2.0 LITERATURE REVIEW

The literature review conducted for this project focuses on history, construction, materials, analysis, and seismic performance of steel frame structures with unreinforced masonry (URM) infill walls.

2.1 Construction History

Builders began using cast and wrought iron construction during the Industrial Revolution. Cast iron is the product of re-melting pig iron, a high carbon content iron, with iron or steel scrap and, although brittle, is considered strong in compression. Wrought iron is a fairly pure iron containing slag from the smelting process, making the iron fibrous (similar in appearance to wood grain) and resulting in a ductile product, considered suitable for welding and similar processes, with a lower carbon content than commercial steel.

Evidence of the early use of iron as an architectural material can be seen in nineteenth-century European railway stations, factories, exhibition buildings, and early department stores. A prime example was the Crystal Palace seen in **Figure B** (below), a monument to British industrial power erected for the 1851 World's Fair.

Figure B: The Crystal Palace (1851), London, England

Source: www.britannica.com

Engineers and architects, such as Viollet le Duc, a proponent of the structural rationalist movement, soon experimented with iron in office and residential construction which allowed the creation of open inner spaces and atria. Iron was also experimented with in tall structures including water towers, grain silos, and, most famously today, the Eiffel Tower which, like the Crystal Palace, was constructed for a World's Fair exhibition. The idea of structural rationalism that Le Duc worked to promote was an architectural movement in which a building's aesthetic composition imitated the load flow and was an honest representation of a building's structural form. The ideas of the structural rationalism movement greatly influenced the architectural evolution of steel construction, especially high-rise buildings, into the twentieth century.

In the United States, structural experimentation with iron in the nineteenth century is most visible in Chicago and New York, where architects worked to create ever higher

structures, typically five to seven stories tall, using early formulations of a skeletal frame structure inspired by the earlier industrial applications of cast iron. A major innovation of early high rise construction was using only the iron frame to resist all loads applied to the structure, first achieved in 1885 with William Le Baron Jenney's Home Insurance Building, as seen in **Figure C** (below).

Figure C: The Home Insurance Building (1885), Chicago, IL
Source: Pridmore and Larson 2005

In this structure, Jenney designed the masonry walls to bear fully on the skeletal frame. Because of this innovation, many consider the Home Insurance Building as the first skyscraper, even when its original ten stories and architectural design were quickly

overshadowed by higher, more refined examples of high rise construction -- many of which were designed in what is considered the Chicago Style, including Burnham and Root's Reliance Building (1895) and Monadnock Building (1891). Both of these buildings are shown in **Figure D** (below).

Figure D: The Reliance Building (1895), Chicago, IL (*left*) and the Monadnock Building (1891), Chicago, IL (*right*)

Source: Pridmore and Larson 2005

The development of the Bessemer process in the early 1850s led to the first practical large-scale use of steel in commercial and technological fields, including the building construction industry. This process, created in England, was the first method to reduce the carbon content of iron for steel production at an industrial scale. The method

removed impurities in pig iron through oxidation within a Bessemer Converter, an eight-to thirty-ton capacity tank like that shown in **Figure H** (below).

Figure E: Bessemer Converter
Source: www.davistownmuseum.org

The Bessemer process created the opportunity for the cheaper, more efficient production of steel, lowering production costs to the rough equivalent of those for wrought iron and eventually prompting manufacturers to produce steel instead. **Figure F** (below) compares the production of steel and iron during the late nineteenth and early twentieth centuries, illustrating this trend. The Bessemer process led to the completion of the first completely steel skeleton in high-rise construction with the sixteen-story Manhattan Building (1890) in Chicago, Illinois, also designed by Jenney and shown in **Figure G** (below).

Figure F: Steel and Iron Production, 1866-1920

Source: voteview.com

Figure G: The Manhattan Building (1890), Chicago, IL

Source: Zukowsky 1987

With the completion of the Manhattan Building and other similar structures, architects quickly recognized the benefits of steel for its ability to create light weight, open structures, maximizing window space and increasing building height capabilities above those permitted by traditional masonry or iron construction. These factors especially appealed to owners and architects trying to maximize retail space within a confined city. During the late nineteenth and early twentieth centuries, Chicago became a center for this transforming face of architecture, as engineers and architects experimented and refined methods of steel and high rise construction (Kostof 1995). New York was a second center for innovation in steel construction, where designers created a central core as part of the steel frame to take a majority of structural loads. The use of a central core system meant that less structural support was needed in the rest of the structure, further increasing floor and window space. Over time, the use of masonry was transformed from a load carrying component of the structure to a material used to create a specific architectural finish, like the Board of Trade Building (1930) and the Field Building (1934), both shown in **Figure H** (below).

Figure H: The Board of Trade Building (1930), Chicago, IL (*left*), and the Field Building (1934), Chicago, IL (*right*)
Source: Pridmore and Larson 2005

This hybrid system first used Chicago during the 1880s dominated high rise construction into the 19030s, quickly migrating to other major cities in the United States, including those in seismic regions like San Francisco and Los Angeles. The first steel framed “skyscraper” in the western United States was the San Francisco Chronicle Building (1889), shown in **Figure I** (below), designed by the Chicago-based firm Burnham and Root.

Figure I: The Chronicle Building (1889), San Francisco, CA
Source: Corbett 1979

By the time of the 1906 San Francisco earthquake, within the city there were thirty completed buildings and ten under construction which were steel framed structures with URM infill walls. A majority of these structures were found in San Francisco's business district, including the Crocker-Woolworth Building shown in **Figure J** (below). Based on damage surveys conducted after the earthquake, these hybrid structures were the most likely to have survived the earthquake and fires intact (Hamburger and Meyer 2006).

Figure J: The Crocker-Woolworth Building (1892), San Francisco, CA

Source: Freeman 1932

The successful seismic performance of steel frame buildings with URM infill walls during the San Francisco Earthquake and greater building heights obtained in New York City with the use of a dense steel frame structural core helped to drive the design and construction of many steel buildings across the United States. This steel construction boom of the early twentieth century continued until 1929, when the Great Depression caused an almost complete stoppage of all construction.

Most new construction did not resume until after World War II, by which time steel technology had greatly evolved, moving from riveted steel frame structures to modern bolted and later, welded, moment-resisting steel frames. An example of this later

generation of steel construction is architect Mies van de Rohe's apartments at 860-880 Lakeshore Drive in Chicago, Illinois (1952) shown in **Figure K** (below).

Figure K: 860-880 Lakeshore Drive (1952), Chicago, IL
Source: Saliga 1990

Since the introduction of steel as a building material in the nineteenth century, high-rise construction has been transformed from masonry clad frames to exposed steel frames, using a range of building materials and construction methods for both infill walls and the surrounding frame.

2.2 Materials and Construction of Infill Walls

Various materials have been used for infill walls, depending on the era of construction. Early steel frame construction generally used multi-wythe or cavity walls of

unreinforced clay brick with sand brick, stone, or terra cotta veneer for exterior walls, prompting the creation of the intricate façades common in the early twentieth century. An example of these early twentieth-century facades is shown by the work of architect Louis Sullivan, pictured in **Figure L** (below).

Figure L: Louis Sullivan Façade
Source: Saliga 1990

Additionally, another form of URM infill, hollow clay tile (HCT), has been used historically as infill material and for modern building facades and interior partitions. Because HCT is a brittle material, it was used mostly in architectural or fireproofing applications. As a fireproofing material, HCT was used historically to create interior pilasters, to cover the structural steel frame, and to create floor systems spanning between joists, such as the system seen in **Figure M** (below).

Figure M: Fireproof Floor System

Source: Rabun 2000

Post-World War II construction typically used steel reinforced clay brick walls or concrete masonry unit (CMU) walls. CMU infill construction can be grouted or ungrouted, reinforced or unreinforced, with strength and ductility dependent on the amount of grout and reinforcing present. UngROUTED infill walls are structurally weak because large in-plane forces can cause compressive splitting of the CMU face shell, which occurs simultaneously with a complete loss of masonry strength. Additionally, in ungrouted construction, sliding-shear resistance only comes from bed-joint mortar. Grouted CMU infill walls have a much higher strength: Under in-plane forces, spalling of the CMU face shell can occur, but the grouted core has a better ability to resist additional loading than ungrouted construction, especially if reinforced.

Additionally, concrete is also used as infill historically and in modern construction. In older construction, engineers typically used steel reinforcing in concrete infill walls only for temperature and shrinkage control and typically did not design

reinforcing to resist structural loads. In contrast, for newer construction well-reinforced concrete infill walls are designed to act compositely with the surrounding frame.

The geometry of infill wall construction varies with length to height aspect ratios ranging from approximately 1:1 to 3:1 and the use of solid panels or panels with openings, between which performance differs considerably. A solid panel completely fills the space within the surrounding frame components. Gaps present at the top or sides of an infill wall, present because of poor construction or concrete shrinkage, can negatively affect a wall's performance. The combined frame and infill wall system will perform as a frame system until those gaps close due to lateral displacement during a seismic event, after which the two parts of the structure act compositely. Although seismic gaps can be designed for use in infill construction, doing so is not a common practice in the United States.

A panel with openings exists generally for door or window perforations as seen in the construction of the San Francisco Merchants Exchange Building (1903) in **Figure N** (below). Opening sizes vary greatly between structures, and the existence of such openings is the condition which most affects the seismic behavior and strength of infill walls. Behavior of perforated walls depends on the size and location of openings: an opening in the center of a wall can cause poor infill performance. Alternatively, a partial height infill is fairly strong, but due to weakness in column shear capacity can cause a short-column mechanism to form in the surrounding frame. Partial width infill walls also exist, in which window openings extend the full height, instead of the full width, of the structural frame.

Figure N: Merchant's Exchange Under Construction (1903), San Francisco, CA
Source: Freeman 1932

Variation in infill wall construction greatly affects the lateral load resisting capabilities of a structure. Understanding the influence different infill wall configurations between buildings or within an individual building is critical in predicting structural performance.

2.3 Materials and Construction of the Structural Frame

For high-rise buildings, infill walls were used in conjunction with steel or concrete frame construction, but older infill wall construction is more commonly found with a steel frame building. With a steel frame, column and beam elements are usually I-sections or wide flange sections, but older structures also are found with built-up

members, circular column sections, and channel or S-sections for beams. Historically, in early iron or steel frame structures, a joist and girder element were usually joined together similarly to light-frame timber construction, with the joist resting on the girder and rivets joining the bottom joist flange and top girder flange. However, the frame could also be connected such that the top of the joist would be flush with the top of the girder. Girder and column elements were typically joined by semi-rigid riveted connections as shown in **Figure O** (below). These connections were created by extending the girder's web plate to the column; alternatively, a seated connection with top restrainers would be used. The extension of the web plate above and below the girder provides vertical shear transfer from the girder to the column, as well as rotational restraint for wind resistance, and is sometimes supplemented by knee braces.

Figure O: Riveted Steel Girder to Column Connections

Source: Hamburger and Meyer 2006

Many steel frames are encased by concrete: the beams as a part of the floor system and the columns as an alternate fireproofing method to terracotta. With concrete-

encased construction, the semi-rigid riveted connections behave as fully restrained connections until the confining concrete cracks due to lateral loading.

Often late nineteenth and early twentieth century steel frames support an exterior URM infill wall at each floor level; often, the frame was built fully or partially embedded in the masonry, allowing for transfer of the masonry weight into the frame. This connection was generally detailed by aligning the interior masonry face with the interior beam flange face. Lintel angles were provided to transfer weight to the steel frame, extending from the exterior face of the beam and column flanges to the outer course of masonry as shown in **Figure P** (below).

Figure P: Lintel Angle Transferring Masonry Weight to Steel Frame

Source: Hamburger and Meyer 2006

Reinforced concrete frames were also used in conjunction with infill wall construction although this application was less common than steel framed construction until after World War II. Concrete frames built after World War II were designed as

either ductile or non-ductile depending on seismic risk of the region, a classification mostly dependent on reinforcing details. Ductile detailing entails well-confined concrete with closely-spaced ties in all members and in connections. A well-confined concrete frame surrounding a weak infill panel suffers less critical damage under lateral loads in comparison to a poorly confined concrete frame. Fully ductile design of frames is rare – in the United States such design is typically only found in western regions, and it is seldom found in conjunction with infill wall construction. This type of design requires well-confined beams with adequate development of bottom reinforcing bars. Ductile columns are designed typically with closely spaced spiral hoops, giving a high shear capacity and displacement ductility, with the infill wall acting as the deformation-limiting component of the frame.

Non-ductile frames are common in low seismic risk regions, and may have deficiencies including weak-column, strong-beam effects; lap splices located in hinging zones; and insufficient transverse reinforcing. Transverse reinforcing exists to add confinement, shear strength, and longitudinal reinforcement stability to a frame. Inadequately reinforced connections of such structures in combination with infill wall construction can leave the beam or column vulnerable to shear failure.

Understanding the particular design of a building's load-bearing frame helps gives insight to the capabilities of that frame, to gravity load flow through the structure, and to interaction between the frame and infill wall panel.

2.4 Seismic and Wind Performance

Before the development of current seismic standards, many buildings in the late nineteenth and early twentieth centuries were either not designed to resist lateral loads or were designed to resist an arbitrary value of wind pressure. No state or national requirements existed for structural design and individual cities generally set rules for design, with major concerns focused on fire resistance rather than seismic or wind resistance. San Francisco did not have any requirements for lateral design until after the 1906 earthquake, whereupon the city building code required buildings over 100 feet high to be designed for a wind force of thirty pounds per square foot (Brunnier 1956).

Given the minimal lateral design of these early twentieth-century structures, it is therefore interesting that professional engineers considered the performance of steel framed structures with URM infill walls during the 1906 San Francisco Earthquake as exceptional, and it quickly became a preferred method of construction in seismic regions. The American Society of Civil Engineers' (ASCE) Committee on Earthquake and Fire Damage to Buildings concluded in a 1907 report on earthquake damage that "The well designed steel frame offers the best solution of the question of an earthquake-proof building, as all the stresses can be cared for" (Hamburger and Meyer 2006). The endorsement by prominent engineers of steel construction through this and similar observations helped influence future development of high rise construction.

As would be expected by current engineers, the Committee also noted problems with the URM infill, stating that "a brick spandrel wall adds little, if any, to the bracing of

a steel frame. Many such walls were cracked badly, and moved on the supporting girder. No reliance should be placed upon them, as they are open to all the objections stated in connection with brick walls in general” (Hamburger and Meyer 2006). In fact, damage to URM infill walls ranged from complete collapse to cracked joints and most likely protected the steel frames from significant damage. The Marston Building at 240 Kearney Street experienced out-of-plane failure of the URM infill walls but was under construction at the time and most likely did not have complete confinement of the walls within the structural frame (Hamburger and Meyer 2006). If these riveted steel structures were not designed to resist lateral loads, the URM infill, as the stiffer part of the building, must have resisted a majority of the lateral loads experienced by the structure.

Another instance of the lateral load capabilities of the combined structural system was noted during a wind storm. New York City’s Empire State Building, seen in **Figure Q** (below), was the subject of a 1938 report written for damage incurred during a wind storm in which wind speeds reached ninety miles per hour. The twenty-ninth and forty-first stories of the 102-story structure cracked during the storm; strain gauges in place in the structure did not note strain in the steel frame until after the masonry cracked – the “nonstructural” infill bore all lateral loads until cracking of the masonry occurred (Langenbach et al. 2006).

Figure Q: The Empire State Building (1931), New York, NY

Source: www.empire.state.ny.us

Similar observations of the remarkable performance of steel frame structures with URM infill walls made in earthquakes after 1906 prompted building code officials across the United States to give steel frame structures lenient design criteria through the mid-twentieth century, influencing materials and design methods used for several generations of high rise construction. Projected repair costs were very low in comparison to other structural types, and many engineers believed a well-designed steel structure would experience little to no damage during an earthquake. Similar structures to those tested during the 1906 earthquake and observed by the ASCE Committee were built into the 1930s. Typical design of these structures continued to neglect infill wall resistance to lateral loads and often gravity loads, with beams designed assuming a simply supported

condition, and with lateral loads designed for resistance within rigid beam-column connections, all factors which proved to affect the seismic performance of a structure.

The belief in the structural superiority of steel buildings was shattered by observations of damaged structures during the 1994 Northridge earthquake, the first California earthquake to prompt major changes in methods of steel construction. A large portion of damage from the Northridge earthquake was contained in recently constructed steel structures and caused emergency measures to be taken addressing seismic design problems of those structures. Additional evidence of performance issues with steel framed construction came from observations of the 1995 Kobe Earthquake in Japan. Based on observations from these two earthquakes, it was found that many late twentieth-century welded steel moment frame buildings in the United States had inadequate seismic detailing. This conclusion also made engineers question the safety of that first generation of steel buildings which had greatly influenced high-rise construction.

However, steel high rise structures constructed between the 1880s and 1930s in the United States tend to distribute lateral loads very differently than more recent structures. Although the structures of this earlier era may be considered to have deficiencies by current engineers, they generally have a high degree of redundancy, while structures built between 1945 and the mid 1990s often have very little redundancy. In an early steel structure, if a connection is weak and displays deteriorating hysteretic behavior, that weak connection is found throughout the entire structure as opposed to being confined within one line of a moment or braced frame. Additionally, the URM infill walls and interior masonry partitions provide additional strength and stiffness to the

steel frame which was not recognized in the original design. Because there is a large distribution of loads and deformation, the demands on individual components of early steel structures is most likely less than what occurs in more modern steel structures (Roeder, Leon, and Preece 1996). Understanding recorded accounts and research into the structural performance of steel frame buildings with URM infill during high winds or earthquakes help to develop an understanding of possible issues within that particular structural system.

3.0 Structural Behavior

While studying recorded performance of steel frame buildings with unreinforced masonry (URM) infill walls is important to understanding possible structural issues, it is also important to understand expected structural behavior of that dual system, especially the interaction between the infill wall and surrounding frame. Although infill walls were not usually designed as structural elements, they do experience stress and strain due to lateral load effects from wind and seismic loads. Due to lateral load effects, there are two main behavior modes of infill wall construction: in-plane and out-of-plane behavior. Because infill walls are usually much stiffer than the surrounding structural frame, infill walls generally attract the lateral story forces associated with those loadings. The combination of infill wall stiffness and structural frame flexibility results in a behavioral advantage gained in the interaction of the structural frame and infill wall (Abrams and Angel 1993).

An infill wall under in-plane lateral loading can be considered as a two-dimensional plane stress element. The surrounding frame acts as a confining element, providing strength and stiffness to the masonry wall. When the frame deflects, it will bear on the upper corner of the infill wall, and stresses will be concentrated diagonally from that bearing point across the panel, forming a diagonal compression strut (Guh and Youssef 1993) as shown in **Figure R** (below).

The stiffness of this system appears to allow an infill wall panel to experience high levels of in-plane drift – tests have shown that an URM infill wall panel can

experience one to two percent lateral drift without collapse (Flanagan and Bennett 1999). Tests completed with a multistory and multibay structure resulted in similar in-plane behavior of infill walls (Mosalam, White, and Ayala 1998). Damage to an infill wall from earlier lateral load demands results in a reduction of initial in-plane stiffness under reloading, but overall performance under secondary in-plane loading appears to be comparable to an undamaged wall (Henderson et al. 2003).

Figure R: Compression Strut Analogy for a Solid Infill Wall

Source: Author

A laterally loaded infill wall will often crack either diagonally in compression, due to sliding shear along bed joints, or both (Abrams and Angel 1993). However, the principal failure mechanisms of in-plane loading are corner crushing and diagonal

cracking as opposed to sliding shear, implying that masonry shear capacity does not govern infill wall capacity and that predicting infill wall behavior as a compression strut is the most accurate representation of infill wall capacity (Henderson et al. 2003). Once mortar joints crack, the compression zones are redistributed into the uncracked portions of the wall, which then act as bracing elements of the building frame, as shown in **Figure S** (below).

Figure S: Compression Strut Analogy for a Cracked Infill Wall
Source: Author

A similar compression strut analogy can be used for a perforated infill wall, where a compression strut will form within each section of the wall, as shown in **Figure T**

(below). Ideally, with a perforated infill wall panel, the infill panel and surrounding frame are deformation compatible (Kariotis and Nghiem 1993).

Figure T: Compression Strut Analogy for a Perforated Infill Wall
Source: Author

An infill wall's out-of-plane behavior depends on the connection of the infill to the surrounding frame. Most early steel structures provided some confinement for the URM infill, a condition which resists out-of-plane load effects through arching action. As the masonry is loaded, hinge formation occurs at each support and at mid-span, as shown by **Figure U** (below).

Figure U: Out-of-Plane Behavior of an Infill Wall

Source: Author

The two wall segments created by this hinge formation will rotate as rigid bodies and develop a compressive axial force, pushing against the supporting frame. If the line of action of this compressive force remains within the infill wall plane and the force does not exceed the masonry's crushing strength, the wall will be able to resist the out-of-plane load. Similarly, the load is resisted in each section of the infill of a perforated wall, with spandrel elements acting horizontally and pilaster elements acting vertically (Guh and Youssef 1993). Due to the arching effects of out-of-plane loading, infill panels can have a high degree of stability under imposed uniform loads or drift loads. As long as an infill wall panel is restrained in-plane, which allows the arch-action to develop, only panels with high height-to-thickness ratios are susceptible to loss of stability under uniform loads (Flanagan and Bennett 1999). Tests of infill wall panels have noted that due to vertical and sometimes horizontal arching action, the infill wall panel usually remains stable after ultimate capacity is reached (Henderson et al. 2003).

Combined in-plane and out-of-plane loading appears to have little effect on infill wall behavior. A loss of stiffness occurs with this combined load effect. Prior in-plane loading may result in higher lateral deflections and some strength decrease under uniform out-of-plane loads, but arch-action is still capable of forming. For slender infill walls, in-plane loading can reduce out-of-plane strength by approximately one-half, but for low height-to-thickness ratios, cracked infill walls can still display high levels of strength (Abrams and Angel 1993). Although a combined load effect may result in failure of the infill wall panel, the infill appears to remain stable and has added in-plane strength (Flanagan and Bennett 1999).

In addition to in- or out-of-plane failure, solid and perforated infill panels may be subject to sliding at bed joints, diagonal tension forces, corner crushing, and shear failure. Portions of perforated infill panels also can act in several separate pieces based on size and location of openings, which affects the surrounding frame's behavior. One effect on the surrounding frame is the possible creation of strong column effects which can cause shear failure in beams, strong beam behavior which can cause short column effects, reducing ductility, or the infill can cause tension yielding in the column. Steel frame components may be subject to flexural yielding, shear yielding, and bolted or riveted connection failure (ATC 1999a).

Additionally, encasement of the steel frame in concrete provides increased stiffness and strength which was not factored into the original structural design. This construction may result in lower demands on pieces of the structure because the deformation, load, and resistance are distributed throughout the building. The strength of

masonry infill panels and interior masonry partitions often exceeds the lateral capacity of the steel frames, which may improve building performance during an earthquake. As observed during the 1995 Kobe Earthquake in Japan, infill walls appear to provide enough strength and stiffness to prevent severe damage or collapse to the steel frame (Roeder, Leon, and Preece 1996). Alternately, however, a URM infill may be very brittle and will deteriorate under small lateral deformations reinforcing the fact that interaction between the two portions of the dual system can be critical to overall building strength. Additionally, local failure could cause a concentration of damage in a particular story of the steel frame during a severe earthquake. When the infill fails, the steel frame will most likely have additional strength and ductility, which allows the structure to take higher loads and displacements.

While most damage is generally expected in infill wall components because they are stiffer than the surrounding steel frame, that frame may also experience some damage. Three behavior modes which are possible sources of damage in the steel frame are flexural yielding, shear yielding, and bolted or riveted connection failure. The flexural yielding of a steel frame is expected in regions next to a rigid connection, which occurs in infill frames generally at the base of fixed-base columns. Flexural yielding in infill construction can lead to plastic hinging and is usually coupled with buckling of the affected steel frame component's compression flange and paint cracking. However, the rotational capacity necessary for plastic hinge formation in steel members is high – even in early steel construction – and most likely will not be reached because infill panels limit the interstory drifts which can occur in a structure. Because of the limit imposed on the

steel frame by the infill panel, most damage from flexural yielding is not structurally significant (ATC 1999a).

While flexural yielding of the steel frame would be related to lateral deformations of the structure, shear yielding occurs due to loads imposed on the frame by the infill wall. If corner crushing occurs in an infill component, the diagonal compression strut's line of action can rotate so lateral load is transferred from the wall into the frame within the column instead of the beam-column connection. This load transfer results in a large shear force at the column's end region. In non-compact steel sections which are not confined by concrete, the extra load in the column can lead to web buckling or significant localized shear deformations in the affected steel frame components. However, in steel frames shear yielding is considered a ductile behavior mode and generally does not result in serious damage (ATC 1999a).

The final behavior mode of concern with early steel frame construction is the failure of bolted or riveted connections. These connections are usually encased in concrete, allowing the joint to behave rigidly until cracking occurs in the concrete which happens under high lateral loads. Additionally, yielding of the connection can occur when there is no confinement of the concrete encasing which causes the steel frame connection to act semi-rigidly. Because the ductility capacity of semi-rigid connections is considered high, this yielding is not considered critical. The presence of infill panels in steel frame construction, particularly the introduction of the diagonal strut force previously discussed puts the beam-to-column connection under high axial tension loads. This additional loading may cause prying of angles used as part of the connection, which can appear to

be serious damage. Even with prying damage present, the ductility capability of the connection is still considered high, with tests showing that the steel frame is still capable of several loading cycles before experiencing a low-cycle fatigue failure. By the time this cyclic failure occurs in the steel frame, the infill panel will have already failed (ATC 1999a). Understanding how a steel frame with URM infill behaves under lateral loading is important to predicting behavior in existing structures once certain levels of damage are reached and to properly model the interaction between the two materials for computer analysis.

4.0 METHODS

This section describes the study conducted with the model of a prototypical multistory steel frame building with unreinforced masonry (URM) infill walls in order to better understand the overall system performance and variation in performance levels in different seismic risk regions.

4.1 Background

Original drawings of eight steel framed structures with URM infill walls were obtained from structural engineering firms with experience working on this structural system. Structural engineers at Nabih Youssef & Associates¹, Forrell/Elsesser Engineers², Degenkolb Engineers³, and Simpson, Gumpertz, and Heger⁴ all helped with access to original architectural and structural drawings of those buildings. All of the drawings were of structures constructed between the late nineteenth and early twentieth centuries in California, with one structure located in Los Angeles and the remaining structures located in the San Francisco Bay region.

Although completeness of the drawing sets varies, a study of the drawings was completed to better understand the architectural and structural design of steel frame structures with URM infill walls, including steel frame member sizes, masonry wall dimensions, story heights, bay widths, and variation in openings.

¹ Nabih Youssef & Associates, 800 Wilshire Boulevard, Suite 200, Los Angeles, CA, 90017

² Forell/Elsesser Engineers, Inc., 160 Pine Street, 6th Floor, San Francisco, CA 94111

³ Degenkolb Engineers, 235 Montgomery Street, Suite 500, San Francisco, CA. 94104

⁴ Simpson, Gumpertz, and Heger, The Landmark @ One Market, Suite 600
San Francisco, CA 94105

4.2 Modeling

Based on drawings of the study's most complete set of architectural and structural drawings, a prototypical building was developed using ETABS as seen in **Figure V** (below). After studying the structural system as discussed in Sections 2.0 and 3.0 as well as all sets of drawings, it was decided that the building used for creating the prototypical model reasonably represented steel frame construction with URM infill in the United States in the late nineteenth and early twentieth centuries. The structure used for developing the prototypical building is a 150' high, twelve-story steel frame structure with URM infill walls completed in 1912. There are multiple bay widths, story heights, and mezzanine levels, as well as a change in plan above the lower stories.

Figure V: 3D View of Prototypical Model

Source: Author, Using ETABS

First, material properties for the model were determined based on values consistent with tests completed on materials from the era (AISC 1953; ASCE 2007) and were modeled as nonlinear materials. For example, the steel property data, seen in **Figure W** (below), was created using property data listed in the AISC (American Institute of Steel Construction) reference *Iron and Steel Beams: 1873 to 1952*, which lists the properties of steel beam and column sections created by different manufacturers and in different eras. The values considered for this project were all expected strength values as opposed to lower bound values as discussed in ASCE 41 because the primary concern of the structure was deformation-controlled, not force-controlled, behavior. Although material properties of the three materials used in this structure – steel, concrete, and masonry – all may differ based on the individual project even if built in a similar construction era, for the purposes of this study, no differences in material properties were studied.

Material Property Data	
Material Name	STEEL
Display Color	Color: [Green Swatch]
Type of Material	☒ Isotropic ☐ Orthotropic
Type of Design	Design: Steel (dropdown)
Analysis Property Data	
Mass per unit Volume	0.
Weight per unit Volume	0.
Modulus of Elasticity	29000.
Poisson's Ratio	0.3
Coeff of Thermal Expansion	0.
Shear Modulus	11153.8462
Design Property Data	
Minimum Yield Stress, Fy	30.
Minimum Tensile Strength, Fu	60.
Cost per Unit Weight	1.
<input type="button" value="OK"/> <input type="button" value="Cancel"/>	

Figure W: Steel Property Input (k-in)

Source: Author, Using ETABS

Second, steel cross-sectional properties were created based on a study of the structural drawings for the building discussed above. With the aim of simplifying the prototypical structure, a uniform column size was used at each story of the model, with dimensions created using typical sizes from the original structure, such as the first-story column shown in **Figure X** (below).

Figure X: First-Story Column Input (in)
Source: Author, Using ETABS

Additionally, three beam sizes were used throughout the prototypical building, based on those from the original building, with dimensions determined from *Iron and Steel Beams: 1873 to 1952* (AISC 1953), like the 12I31.5 beam section in **Figure Y** (below). Only primary frame members of the structural system were modeled.

I/Wide Flange Section

Section Name 12I31.5

Properties

Property Modifiers

Material STEEL

Dimensions

Outside height (t3) 12.

Top flange width (t2) 5.

Top flange thickness (tf) 0.544

Web thickness (tw) 0.35

Bottom flange width (t2b) 5.

Bottom flange thickness (tfb) 0.544

Display Color ☐

Figure Y: 12I31.5 Beam Input (in)
Source: Author, Using ETABS

Infill walls were modeled as diagonal compression struts as discussed in Section 3.0. Cross-sectional properties for the equivalent struts representing the unreinforced masonry (URM) infill walls were created using infill dimensions from the original structure and the guidelines described in ASCE 41. Section 7.4 of ASCE 41 focuses on masonry infills, including calculating in-plane stiffness based on an equivalent strut depth determined by the material properties and geometry of an individual infill wall. The strut width used in this process is the actual width of the structure's infill wall: for the first two stories of the prototypical model, the width is 17" (three wythes of brick) while higher stories have a 13" width (two wythes of brick). Along with the actual thickness of the infill wall, the equation for strut depth is used to create the equivalent strut dimensions.

Using this guideline, strut depth (a) is calculated as:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}, \quad \text{Eq. 1}$$

where $\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}} \right]^{\frac{1}{4}},$

- h_{col} is the column height between centerlines of beams (in),
- h_{inf} is the height of infill panel (in),
- E_{fe} is the expected modulus of elasticity of frame material (ksi),
- E_{me} is the expected modulus of elasticity of infill material (ksi),
- I_{col} is the moment of inertia of column (in⁴),
- L_{inf} is the length of infill panel (in),
- r_{inf} is the diagonal length of infill panel (in),
- t_{inf} is the thickness of infill panel and equivalent strut (in),
- θ is the angle whose tangent is the infill height-to-length aspect ratio (radians), and
- λ_1 is the coefficient used to determine equivalent width of infill strut.

Through this process, an equivalent strut depth was calculated for each infill wall and input as a rectangular section in the computer model like the cross-section for a first-story strut shown in **Figure Z** (below).

Figure Z: First-Story Equivalent Strut Input (in)

Source: Author, Using ETABS

The depths of equivalent struts for infill walls with openings were determined assuming a relationship between expected behavior and infill wall area. Based on results from the work of Khalid M. Mosalam, R.N. White, and G. Ayala (Mosalam, White, and Ayala 1998), infill wall strength depends on the area of the opening in comparison to the total infill area. Their research was conducted for creating and executing a practical testing method for multi-story infilled frames using steel frame with unreinforced concrete masonry unit (CMU) infill wall construction. For their study, openings were created in some of the infill walls as part of the test setup and were subjected to the same drift levels. Lateral loading results showed a difference in the force capacity between the solid and perforated infill construction of approximately twenty-five percent – which corresponds to an opening approximately twenty-five percent of the total infill wall area.

For the prototype building created for this project, the relationship between lateral load carrying behavior and opening size allowed the approximation of equivalent strut depths for perforated infill walls in the prototypical building.

Additionally, a concrete shell area element, which provides in- and out-of-plane stiffness, as shown in **Figure AA** (below) was created to represent the floor system. The membrane thickness of 5" was chosen based on the actual thickness of the structural concrete floor in the structure used to develop the prototypical building. The membrane thickness is used in ETABS to determine the stiffness of the shell elements. For consistency, the bending thickness, used for determining bending stiffness of the shell element, was also assigned as 5".

The image shows a software dialog box titled "Wall/Slab Section". It contains the following fields and options:

- Section Name:** DIAPHRAGM
- Material:** CONC (dropdown menu)
- Thickness:**
 - Membrane:** 5
 - Bending:** 5
- Type:**
 - ☒ Shell
 - ☐ Membrane
 - ☐ Plate
 - ☐ Thick Plate
- Load Distribution:**
 - ☐ Use Special One-Way Load Distribution
- Buttons:** Set Modifiers..., Display Color (with a pink color swatch), OK, and Cancel.

Figure AA: Shell Element Input (in)
Source: Author, Using ETABS

A simplified connection system was created to translate fixity in the original structure to a prototypical model. Fixed connections were assigned to columns at the base of the structure, mimicking an actual structure's connection to the foundation. Beam-to-column connections were also fully fixed, mimicking the strength of a concrete encased steel frame, common with this era of construction, although connections in the bare frame were more likely to be partially fixed, as previously discussed in Sections 2.0 and 3.0 and shown in the detail in **Figure BB** (below). In order to imitate the in-plane behavior of the infill wall discussed in Section 3.0, the equivalent struts were pin-connected to the surrounding frame.

Figure BB: Typical Beam-to-Column Connections

Source: Nabih Youssef & Associates

Dead and live gravity loads determined from the construction of the prototype building were applied as area loads to the concrete shell elements and, to represent the weight of brick walls simplified to equivalent struts, line loads were applied to the exterior wall lines. Similarly, area masses were applied to the concrete shell elements and line masses were applied to the exterior wall lines.

Because the affect of lateral loading in the prototypical building was most critical at the exterior frame lines, rigid diaphragms were used to model the lateral displacement of each floor level as opposed to modeling semi-rigid behavior, which would have determined rigidity based the stiffness of objects in an individual diaphragm. A rigid diaphragm was assigned at each floor level as shown by the roof diaphragm in **Figure CC** (below).

Figure CC: Roof Diaphragm
Source: Author, Using ETABS

To verify the nonlinear behavior of hinge definitions used throughout the prototypical building, a single-bay test model was created using steel cross-sections and an equivalent strut with nonlinear hinges. Components of this structure were based on testing completed by Roger D. Flanagan, R.M. Bennett, and J.E. Beavers on steel-frame subjects with hollow clay tile (HCT) infill panels. Their study was undertaken to better understand the behavior of HCT infill under in- and out-of-plane loading (Flanagan, Bennett, and Beavers 1993). In this study, several test subjects were created using varying steel frame members, infill aspect ratios, and connection between frame and infill components. For verification of the prototypical building created for this master's project, one test subject from Flanagan, Bennett, and Beavers' study was modeled in ETABS and calibrated to the results of an in-plane hysteresis load applied to the frame. Complete documentation of this calibration and the prototypical building are found in Appendix B.

Nonlinear hinges were developed for portions of the structure expected to experience nonlinear behavior based on the calibration discussed above. Within the prototypical model, nonlinear behavior is expected in the exterior frame lines (where the URM infill walls exist) because the infill walls will attract a higher percentage of lateral loads than the bare steel frame. For each of the line elements – steel beams, steel columns, and masonry struts – nonlinear hinge properties were developed based on guidelines in ASCE 41. For a steel cross-section, there are guidelines to create a nonlinear response curve taking into account strength degradation; however, for the equivalent struts of a masonry infill wall, there are only guidelines for creating a curve with a constant post-yield stiffness. In order to have a compatible nonlinear model, the

simplified nonlinear static procedure per ASCE 41 was used to model the entire structure, meaning that the nonlinear behavior of steel and masonry were modeled without any stiffness degradation. Beam elements were modeled with moment hinges, column elements were modeled with axial-moment hinges, and equivalent struts were modeled with axial hinges. For the axial-moment hinge in ETABS, a P-M diagram was also defined. Default yield moment and rotation values calculated by the computer program were determined to be accurate and were used to define the yield point of steel beams and columns in the model. However, ETABS calculated yield force values for equivalent strut sections were not accurate so a calculated yield force was input for each strut section.

ETABS also permits the definition of acceptance criteria values in terms of plastic displacement for Immediate Occupancy, Life Safety, and Collapse Prevention performance levels – each which correspond to an expected damage level in a structure during an earthquake. Immediate Occupancy refers to a level with very limited structural damage, Life Safety refers to a level with damage to structural elements but without any collapse, and Collapse Prevention refers to a level with high levels of damage, including significant loss in stiffness and strength, but without collapse (ASCE 2007). For the prototypical building, the values assigned to these three levels were determined using ASCE 41 recommendations – for steel beams, those definitions were for the three performance levels as per Chapter 5 of ASCE 41. For equivalent struts, Section 7.4 of ASCE 41 only gives acceptance criteria for the Life Safety performance level and defines that level as a percentage of the drift ratio as opposed to plastic displacement. Because of this difference, the acceptance criteria defined for equivalent struts in ETABS were set so

that they would not interfere with the model's behavior – that performance level criteria in strut members would not influence the model's behavior. An individual strut's actual performance was then hand calculated by comparing the drift experienced at each level of the model with the yield drift at each step of the nonlinear analysis. The resulting definitions from this modeling process are shown for each type of element – steel beam, steel column, and masonry strut –in **Figure DD**, **Figure EE**, and **Figure FF** (below).

Frame Hinge Property Data for 12I315 - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0
A	0	0
B	1	0
C	1	1000
D	1	1000
E	1	1000

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☒ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	1.	
Life Safety	6.	
Collapse Prevention	8.	

OK
Cancel

Figure DD: Steel Beam Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C1 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0
A	0	0
B	1	0
C	1	1000
D	1	1000
E	1	1000

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative
☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative
Life Safety Positive Negative
Collapse Prevention Positive Negative

Axial Load - Displacement Relationship

☐ Proportional to Moment - Rotation
☒ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure EE: Steel Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for S111 - P

Edit

Point	Force/SF	Disp/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0
A	0	0
B	1	0
C	1	1000
D	1	1000
E	1	1000

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Force and Disp

☐ Use Yield Force Force SF Positive Negative
☒ Use Yield Disp Disp SF Positive Negative

Acceptance Criteria (Plastic Disp/SF)

Immediate Occupancy Positive Negative
Life Safety Positive Negative
Collapse Prevention Positive Negative

Type

☒ Force - Displacement
☐ Stress - Strain
Hinge Length
☐ Relative Length

OK
Cancel

Figure FF: Equivalent Strut Nonlinear Hinge Definition
Source: Author, Using ETABS

Once the prototypical building was modeled in ETABS as discussed, an analysis to better understand the performance of steel-framed structures with unreinforced masonry (URM) infill at different levels of seismic risk could be undertaken.

4.3 Analysis

Analysis of the prototypical building was conducted following provisions in ASCE 41 for linear dynamic and nonlinear static procedures. A linear dynamic analysis using the modal method discussed in section 3.3.2 of ASCE 41 was completed in order to determine the building's elastic fundamental period, T_i . Additionally, a simplified nonlinear static procedure, or pushover analysis, was performed in ETABS on the prototypical building, following section 3.3.3 of ASCE 41, resulting in target displacement predictions and a pushover curve in each major building direction. The first portion of this analysis was the calculation of target displacements using site-specific information for each of the seventeen cities used in this study. The calculation used to determine target displacement is presented in section 3.3.3.3.2 of ASCE 41 as:

$$\delta_i = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g, \quad \text{Eq. 2}$$

where $T_e = T_i \sqrt{\frac{K_i}{K_e}},$

- C_0 is the modification factor to relate spectral displacement of an equivalent single-degree of freedom system to the roof displacement of the building multi-degree of freedom system,
- C_1 is the modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response,

- C_2 is the modification factor to represent the effects of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response,
- S_a is the spectral response acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration,
- T_e is the effective fundamental period of the building in the direction under consideration (seconds),
- g is the acceleration of gravity,
- T_i is the elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis (seconds),
- K_i is the elastic lateral stiffness of the building in the direction under consideration (k/in), and
- K_e is the effective lateral stiffness of the building in the direction under consideration (k/in).

The cities used for target displacement calculations have 1880-1930s steel framed construction with URM infill walls and geographically lie in a range of low, medium, and high seismic risk regions. Because the computer model represented a prototypical structure as opposed to a specific building, a major focus of the nonlinear analysis was to study expected building performance in different seismic risk zones. The two pushover curves from this nonlinear analysis can be compared to calculated target displacement values, allowing discussion of possible issues within each seismic risk zone and giving insight to possible structural issues within this building type in different locations.

5.0 DATA AND RESULTS

A prototypical model seen in **Figure GG** (below) was developed in ETABS based on the process described in Section 4.0. Once the model was completed, modal and nonlinear static analyses were conducted, and target displacements were calculated as discussed in Section 4.0 for comparison with pushover analysis results.

Figure GG: 3D View of Prototypical Model
Source: Author, Using ETABS

The first procedure, the modal analysis, showed an expected pattern with the first mode acting along the model's y-axis as shown in **Figure HH** (below, first), the second mode along the x-axis as shown in **Figure II** (below, second), and the third mode acting

as a torsion mode as shown in **Figure JJ** (below, third). Results of the first six modes from the modal analysis are shown in **Table A** (below, fourth).

Figure HH: First Mode Shape, Along Grid Line J (Y-Axis)
Source: Author, Using ETABS

Figure II: Second Mode Shape, Along Grid Line 1 (X-Axis)
Source: Author, Using ETABS

Figure JJ: Third Mode Shape, Along Grid Lines 1 (left) and J (right)
Source: Author, Using ETABS

Mode	Period	Percentage of Mass Participation	
		Rx	Ry
1	2.106345	96.7586	1.6623
2	1.942977	98.7234	98.323
3	1.207895	99.4372	99.7951
4	0.726588	99.6292	99.8089
5	0.70194	99.7149	99.88
6	0.489715	99.7228	99.9004

Table A: Modal Analysis Results
Source: Author

The second analysis, a nonlinear static analysis was completed in two parts: target displacement predictions and pushover analysis. Target displacements were determined using the equation discussed in Section 4.0. For the prototype building, $C_0 = 1.5$, $C_1 = 1.0$, $C_2 = 1.0$, S_a varies based on region of seismic risk, $T_e = 2.169$ seconds along the x-axis and 1.988 seconds along the y-axis, and $g = 386.4 \text{ in/s}^2$. S_I and S_S values, the spectral response acceleration parameter at a one-second period and the spectral response

acceleration parameter at short periods, respectively, both necessary for calculating S_a , were determined using a United States Geological Survey (USGS) spectral response acceleration mapping tool. Due to a lack of site-specific soil data for all cities used in this parametric study, a standard soil site class D, which corresponds with a stiff soil, was used for all of the calculations as per ASCE 41 (1.6.1.4). Because only one soil profile was used, locations with known poor quality soils, including Chicago and San Francisco, may actually have higher target displacements than calculated for this study – this issue is further discussed later in this section as well as in Section 6.0. Results of the target displacement calculations for seventeen United States cities are shown in **Figure KK** and **Figure LL** (below) for the x- and y-axes of the prototype building.

Figure KK: Comparison of Target Displacements Between Cities: X-Direction
Source: Author

Figure LL: Comparison of Target Displacements Between Cities: Y-Direction
Source: Author

Looking at the target displacement values in the x-direction and y-direction, it appears that a similar response is expected for the structure along either axis, with the largest difference between values seen in the highest target displacement locations, which were calculated using Eq. 2 as Los Angeles and San Francisco.

Based on a comparison of the results of the target displacement calculations for different cities, five main groups relating to the expected displacement of the structure in each city can be created; these groups appear to correlate to seismic risk zones as displayed in a spectral response map of the United States seen in **Figure MM** (below).

Figure MM: Map of United States Cities Used for Study with Seismic Risk Regions
Source: Author

The cities in the first group (Group 1) – Baltimore, Chicago, Cleveland, Detroit, Minneapolis, and Washington, DC – all lie in what are considered low seismic risk regions, with calculated target displacements along both main axes less than 5" (0.28% drift). The second group (Group 2) – Atlanta, Birmingham, Boston, and New York City – appears to be a slightly higher risk seismic group, with target displacements in both the x and y direction ranging from approximately 5" to 8" (0.28% to 0.44% drift). The third group (Group 3) – Nashville and St. Louis – includes cities in a moderate seismic risk region, with target displacements ranging between 8" and 12" (0.44% to 0.67% drift). The fourth group (Group 4) – Charleston, Portland, and Seattle – is a moderate-to-high

risk seismic group with target displacements ranging between 17" and 25" (0.94% to 1.39% drift). The final group (Group 5) – Los Angeles and San Francisco – includes cities known to be in high risk seismic regions with target displacements lying between 30" and 40" (1.67% to 2.22% drift).

Results from target displacement calculations for the seventeen different cities land in five possible groups related to known seismic risk zones; however, an issue with the results of these calculations occurs due to an assumption made for computation purposes. While target displacements for the cities are mostly what is expected based on the major fault zones of the United States, at least two cities' calculated target displacement values are possibly lower than expected. Both San Francisco and Chicago are known to have poor soil profiles but for the purposes of this study, both sites were assumed to have a fairly stiff soil profile; the result of this assumption is that target displacements determined for both San Francisco and Chicago may be lower values than are actually expected for each region.

For example, the Marina district of San Francisco – notorious for the massive areas which suffered from soil liquefaction during the 1989 Loma Prieta earthquake – is on an alluvial soil bed, a Class F according to section 1.6.1.4.1 of ASCE 41, which requires site-specific soil evaluation to determine spectral response parameters. Similarly, Chicago was built on existing swamp land – with a similar soil profile to peat or organic clay – and is also classified as a soil site Class F. The poor soils grouped into the Class F definition all correspond with higher damage levels under seismic loading. This discrepancy means that if, based on an individual site's soil profile, the target

displacement is higher than predicted by this study, force demands on the structure also increase.

The final portion of the analysis, the pushover analysis, was completed for each major axis of the prototype structure. The structure was first analyzed by ignoring P- Δ effects and then with P- Δ effects. The addition of P- Δ effects to the analysis lowered the post-yield stiffness of the structure, as seen in **Figure NN** (below), and is used for the remainder of the analysis in the project.

Figure NN: Force-Displacement Curves With and Without P- Δ Effects

Source: Author

A comparison of each of these computer-generated curves was made with approximate force-displacement relationships calculated using a bilinear approximation from ASCE 41 (3.3.3.2.5) and a linear hand approximation. The comparison of the three curves for each building axis is seen in **Figure OO** and **Figure PP** (below). For reference, pushover curves comparing force with building drift in each major direction are included in **Figure QQ** and **Figure RR** (below).

The ASCE 41 bilinear approximation is an idealized force-displacement curve used to calculate the effective lateral stiffness which, in turn, is used to determine the structure's effective fundamental period using procedures described in ASCE 41. The linear hand approximation was used as a verification of the nonlinear results and took into account the lateral stiffness contribution of the prototypical model's columns and struts. For this approximation, the lateral stiffness and yield force of each strut and column was calculated and a total stiffness and yield force were determined for each story level. Using a triangular load distribution, the lateral force required to cause one story to yield was determined. A deflection corresponding to that level of stiffness and lateral load was calculated, resulting in a yield-point to compare with results from the computer analysis. Using this method, it was determined that the prototypical building would first experience yielding at the fourth story, which was the stiffest level, along both the x- and y-axes, the same result which occurred in the ETABS computer analysis.

Figure OO: Force-Displacement Curves: X-Direction

Source: Author

Figure PP: Force-Displacement Curves: Y-Direction

Source: Author

Figure QQ: Force-Drift Curves: X-Direction

Source: Author

Figure RR: Force-Drift Curves: Y-Direction

Source: Author

In the x-direction, the first point on the ETABS force-displacement curve occurred when one diagonal strut reached its first yield. The second point – where the “ETABS Output” curve appears to reach a yielding point – formed when forty-five diagonal struts formed hinges (first yielding) and twenty-eight diagonal struts exceeded the Life Safety performance level. The third point occurred when forty-eight diagonal struts exceeded their first yielding points and 116 diagonal struts exceeded the Life Safety performance level. At the fourth point, four diagonal struts exceeded first yielding, 179 diagonal struts exceeded the Life Safety performance level, and four beam hinges yielded. At the fifth point, two diagonal struts exceeded first yielding, 192 diagonal struts exceeded the Life Safety performance level, 123 beam hinges exceeded first yielding, and 81 beam hinges exceeded the Immediate Occupancy performance level. At the sixth point, one diagonal strut exceeded first yielding, 198 diagonal struts exceeded the Life Safety performance level, fifteen beam hinges exceeded first yielding, 256 beam hinges exceeded the Immediate Occupancy performance level, twenty-six beam hinges exceeded the Life Safety performance level, eight beam hinges exceeded the Collapse Prevention performance level, twelve column hinges exceeded first yielding, and four column hinges exceeded the Immediate Occupancy performance level. At the seventh point, 212 diagonal struts exceeded the Life Safety performance level, twenty-six beam hinges exceeded first yielding, 176 beam hinges exceeded the Immediate Occupancy performance level, seventy-nine beam hinges exceeded the Life Safety performance level, sixty-six beam hinges exceeded the Collapse Prevention performance level, one column hinge exceeded first yielding, and twenty-two column hinges exceeded the

Immediate Occupancy performance level. At the final point, 213 diagonal struts exceeded the Life Safety performance level, twenty-five beam hinges exceeded first yielding, 109 beam hinges exceeded the Immediate Occupancy performance level, 102 beam hinges exceeded the Life Safety performance level, 121 beam hinges exceeded the Collapse Prevention level, two column hinges exceeded first yielding, thirteen column hinges exceeded the Immediate Occupancy level, and nine column hinges exceeded the Life Safety performance level. The summary of the pushover data can be seen in **Table B** (below).

Number of Hinges Surpassing Each Performance Level									
	Diagonal Struts		Beams				Columns		
Step	Yield	L.S.	Yield	I.O.	L.S.	C.P.	Yield	I.O.	L.S.
1	1	0	0	0	0	0	0	0	0
2	45	28	0	0	0	0	0	0	0
3	48	116	0	0	0	0	0	0	0
4	4	179	4	0	0	0	0	0	0
5	2	192	123	81	0	0	0	0	0
6	1	198	15	256	26	8	12	4	0
7	0	212	26	176	79	66	1	22	0
8	0	213	25	109	102	121	2	13	9

Table B: Force-Displacement Event Summary: X-Direction

Source: Author

The final step of the pushover is seen in plots of hinge formation of the structure in **Figure SS** and **Figure TT** (below).

Figure SS: Hinged Structure, X-Axis Pushover, 3-D View
 Source: Author, Using ETABS

Figure TT: Hinged Structure, X-Axis Pushover, Grid Lines 1 (*left*) and 8 (*right*)
 Source: Author, Using ETABS

In the figures above, circles represent hinges which surpassed the first yield point.

In the y-direction, the first point on the ETABS force-displacement curve occurred when one diagonal strut reached its first yield. The second point – where, similarly to the x-direction, the “ETABS Output” curve appears to reach a yielding point – formed when forty-seven diagonal struts formed hinges (first yielding). The third point occurred when forty-eight diagonal struts exceeded their first yielding points and thirty-three diagonal struts exceeded the Life Safety performance level. At the fourth point, two diagonal struts exceeded first yielding, 134 diagonal struts exceeded the Life Safety performance level, and seventeen beams hinges yielded. At the fifth point, 151 diagonal struts exceeded the Life Safety performance level, fifty-two beam hinges exceeded first yielding, and 140 beam hinges exceeded the Immediate Occupancy performance level. At the sixth point, 169 diagonal struts exceeded the Life Safety performance level, eighteen beam hinges exceeded first yielding, eighty-one beam hinges exceeded the Immediate Occupancy performance level, ninety beam hinges exceeded the Life Safety performance level, fifty-five beam hinges exceeded the Collapse Prevention performance level, eighteen column hinges exceeded first yielding, seven column hinges exceeded the Immediate Occupancy performance level, one column hinge exceeded the Life Safety performance level, and three column hinges exceeded the Collapse Prevention performance level. At the seventh point, 180 diagonal struts exceeded the Life Safety performance level, nine beam hinges exceeded first yielding, seventy-two beam hinges exceeded the Immediate Occupancy performance level, twelve beam hinges exceeded the Life Safety performance level, 172 beam hinges exceeded the Collapse Prevention performance level, two column hinges exceeded first yielding, twenty-six column hinges

exceeded the Immediate Occupancy performance level, and eighteen column hinges exceeded the Collapse Prevention performance level. At the final point, 180 diagonal struts exceeded the Life Safety performance level, six beam hinges exceeded first yielding, forty-four beam hinges exceeded the Immediate Occupancy performance level, fifty-one beam hinges exceeded the Life Safety performance level, 174 beam hinges exceeded the Collapse Prevention level, one column hinge exceeded first yielding, nineteen column hinges exceeded the Immediate Occupancy level, seven column hinges exceeded the Life Safety performance level, and twenty-two column hinges exceeded the Collapse Prevention performance level. A summary of the steps of hinge formation are contained in **Table C** (below).

Number of Hinges Surpassing Each Performance Level										
Step	Diagonal Struts		Beams				Columns			
	Yield	L.S.	Yield	I.O.	L.S.	C.P.	Yield	I.O.	L.S.	C.P.
1	1	0	0	0	0	0	0	0	0	0
2	47	0	0	0	0	0	0	0	0	0
3	48	33	0	0	0	0	0	0	0	0
4	2	134	17	0	0	0	0	0	0	0
5	0	151	52	140	0	0	0	0	0	0
6	0	169	18	81	90	55	18	7	1	3
7	0	180	9	72	12	172	2	26	0	18
8	0	180	6	44	51	174	1	19	7	22

Table C: Force-Displacement Event Summary: Y-Direction

Source: Author

The final step of the pushover is seen in plots of the structure's hinge formation shown in **Figure UU** and **Figure VV** (below).

Figure UU: Hinged Structure, Y-Axis Pushover, 3-D View
Source: Author, Using ETABS

Figure VV: Hinged Structure, Y-Axis Pushover, Grid Lines A (left) and J (right)
Source: Author, Using ETABS

In the figures above, circles represent hinges which surpassed the first yield point.

In the ETABS model, the first hinges to yield in either direction were at the fourth story, followed by hinge formation in other lower stories of the structure and with the

structural yield point occurring at the yield of the forth story. This event is similar to the response seen in the hand approximation – the first yielding of the structure occurs at the forth story, which has a higher stiffness than some stories below it in the model. This higher stiffness value in the fourth story, which is similar in strength to the first story, means that the forth story attracts higher lateral load levels, forcing it to yield first. Additionally, similarity between the hand approximation and the ETABS Output curves implies accuracy in the computer model. Although the hand approximated curve has a lower initial stiffness and experiences yielding later than the structural model, it is an approximation and does not take into account all elements adding stiffness to the structure such as beams and diaphragm elements. A source of inaccuracy in the prototypical building's pushover curves is that after Life Safety ductility performance levels are met for the masonry struts, the model does account for the associated stiffness change of an infill element, implying that at higher displacements, the pushover curve is not conservative.

The comparison of those pushover curves with the target displacement calculations is the final result of the nonlinear static analysis and is necessary to evaluate the relationship between seismic risk zone and expected damages to a steel framed building with URM infill walls. Graphic representations of this comparison are shown in **Figure WW** and **Figure XX** (below).

Figure WW: Comparison of Pushover Curve with Target Displacements: X-Direction
Source: Author

Figure XX: Comparison of Pushover Curve with Target Displacements: Y-Direction
Source: Author

Based on this comparison, it appears that the prototypical building placed in any city used for this study would be able to reach expected target displacements, however, Groups 5 cities, which include cities in the highest risk seismic zones, would be unable to reach the expected target displacements until after some steel beam hinges in the structure surpass collapse prevention performance demands. Also, as previously discussed, when target displacement levels are reached for Group 4 and 5 cities, the pushover curve is not conservative because the ductility capacity of most masonry struts in the prototypical building have exceeded Life Safety performance criteria. This affect implies that overall structural stiffness is higher in the model than would actually exist. Cities in the three lower risk groups – Groups 1, 2, and 3 – would be expected to allow the structure to reach predicted target displacements under much lower force demands, but only after initial yielding occurs in the prototypical building. The prototypical structure experiencing target displacement demands for all Group 1 cities except Detroit and Minneapolis – the lowest seismic risk cities – as well as any Group 2 or Group 3 city, would be relying on the post-yield strength of the structure with all equivalent struts at the fourth story of the prototypical building having reached Life Safety performance levels, as previously discussed. The high levels of demand placed on the prototypical building to reach target displacement predictions – especially in low risk seismic zones – is important in better understanding the performance expectations and capabilities of steel frame structures with URM infill.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Understanding expected damage for a particular structural system within a specific seismic risk zone is critical for approaching any existing building project. In this way, the comparison between the computer model force-displacement curves and calculated target-displacement values is important in leading to conclusions about the performance of late nineteenth- and early twentieth-century steel framed structures with unreinforced masonry (URM) infill walls.

6.1 Conclusions

The most interesting portion of the project for practicing engineers may be a comparison between the predicted target displacements calculated for several cities and the nonlinear pushover curves, as shown in Section 5.0 by **Figure WW** and **Figure XX** (page 69). Evaluating the corresponding physical behavior the structural system undergoes at differing performance levels is important in creating an understanding of potential structural issues in varying seismic risk regions. Performance issues and related damage for steel frame structures with URM infill differ between seismic risk zones, a fact which is summarized in Appendix A, the recognition of those differences will help structural engineers formulate seismic rehabilitation measures necessary for varying cities.

First, in Group 1 cities – Baltimore, Chicago, Cleveland, Detroit, Minneapolis, and Washington, DC – as may be expected for very low seismic risk regions, the prototypical structure meets Life Safety requirements as defined by ASCE 41. However,

although the prototypical building performs fairly well, there is still a danger of possible falling URM bricks in the lower stories of the building. The danger for significant damage exists because in reaching target displacement levels, the prototypical building experiences post-yield force and displacement demands in all locations except Detroit and Minneapolis. The prototypical building in the remaining Group 1 cities would be expected to undergo initial yielding of many infill components, including the yielding of the entire fourth story due to diagonal strut demands. Physically, the extent of damage expected for the prototypical building or a similar structure in a Group 1 city includes substantial cracking of URM infill panels concentrated in the lower stories of the building and the possibility of falling bricks, especially in the stiffest levels of lower stories within the building. Although significant damage may be present in the structural system's infill walls, there is no expected damage to the surrounding steel frame. Because of the possible hazards in these low risk regions, seismic strengthening may be necessary for infill walls, especially in lower stories of similar structures, in order to avoid hazards of falling brick in the existing masonry construction.

Second, Group 2 cities – Atlanta, Birmingham, Boston, and New York City – are in slightly higher seismic risk regions than Group 1 cities and are still able to meet Life Safety Requirements defined by ASCE 41. The prototypical building in these areas is still at risk for possible falling URM bricks in the lower stories of the structure as well as being at risk for collapse of URM panels in that same part of the structure. Hazards exist because the prototypical building or a similar existing structure in these seismic regions would be experiencing significant stiffness loss, including cracked walls due to diagonal

strut demands in the infill components. Extent of damage for the prototypical building includes cracking throughout the structure but is most severe in the lower stories.

Additionally, there is the possibility of bricks falling in the building's lower stories and the possibility of entire panels falling in the lower, stiffer stories of the structure.

However, while significant damage will occur in the infill panels when critical displacement demands are met in Group 2 cities, there is no expected damage for the surrounding steel frame. Similar structures to the prototypical building in these seismic risk regions should have infill panels retrofitted, particularly in middle and lower stories, to avoid falling hazards from brick elements of the structure. In order to determine the critical locations for these repairs, some detailed seismic evaluation may be necessary.

Unlike in Group 2 cities, once target displacement levels are reached for Group 3 cities – Nashville and St. Louis – the prototypical building does not meet Life Safety requirements defined by ASCE 41. This classification includes moderate seismic risk zones, with target displacements only slightly higher than in Group 2 cities, but expected performance changes considerably between the two groups. The prototypical building in Group 3 regions is at risk for possible collapse of URM bricks and URM panels throughout the lower stories of the structure, damage levels corresponding to the failure of many infill panels to meet Life Safety performance goals, especially in lower stories. Damage to the prototypical building or a similar structure in moderate seismic risk regions would include severe cracking of masonry construction throughout the building. Additional damage, in all but the upper stories of the structure, would exist as falling bricks or panels. Even with this severe level of damage present in the masonry

components of the structure, no significant damage would be expected in the steel frame. The hazards created by falling masonry make seismic evaluation necessary for structures similar to the prototypical building in these moderate seismic regions.

The increase in Life Safety hazards between in the transition between the Group 3 cities and Group 4 cities – Charleston, Portland, and Seattle – is significant, with ASCE 41 Life Safety performance goals not met for strut elements throughout the structure, as opposed to being confined to the lower stories. Deficiencies in meeting those performance goals include falling URM bricks and collapsed masonry panels at all stories of the building. Additionally, beam components within the steel frame will exceed Immediate Occupancy performance levels but damage is not as critical as in the masonry panels and complete structural collapse of steel framing is not expected in the building. Extent of damage to the prototypical building or a similar structure would most likely include severe masonry cracking throughout the building, falling bricks, and falling panels. Steel frame damage does exist within the beam components, specifically in the middle and lower stories of the structure, including visible cracking of concrete encasement, flaked paint on steel sections, and possible beam-column connection prying. Because this era of construction did not have design of plastic hinges, damage to the steel frame is expected to occur at or near the connections instead of along the length of the beam. Seismic strengthening of a structural system similar to the prototypical building would be required in the infill panels to prevent falling hazards from the masonry construction and further study would be necessary to better predict the force-displacement behavior of the structure once Life Safety performance levels are not met

throughout the structure because, as discussed in Section 5.0, those results are not conservative for masonry elements at high levels of drift. Additional evaluation and strengthening may be required in order to prevent significant damage to the steel connections.

The prototypical building in a Group 5 city – Los Angeles and San Francisco – would perform very similarly to what is expected in a Group 4. As may be expected for the highest risk regions, the prototypical structure is unable to meet Life Safety performance levels defined in ASCE 41. The structure will be inadequate to meet that performance level due to hazards from falling URM bricks and URM wall panels throughout the structure. Severe damage is also expected in the steel frame, with both beam components in the structure exceeding Life Safety performance levels and in some cases exceeding Collapse Prevention performance requirements. Additional damage would exist in the columns with columns at the base of the structure exceeding Immediate Occupancy performance levels. Damage to a structure similar to the prototypical building under the expected demands of a Group 5 earthquake would include severe masonry cracking throughout the structure, falling bricks, and falling panels. Additional damage would exist in steel beam and column components as visible cracking of concrete encasement, flaked paint on steel sections, and possible beam-column connection prying. Severe damage will also be present in beam components, visible as local beam buckling, extensive beam distortion, and connection fractures. Seismic evaluation and strengthening of a structure's masonry components and steel frame components would be required in this region. Further analysis would be required in this

region to better predict behavior of infill wall components after they exceed Life Safety performance demands and to better predict the force-displacement behavior of the structural system.

While the insufficient overall capacity for Life Safety performance demands may be expected by a modern engineer working on a structure in a Group 4 or Group 5 region, the highly regarded behavior of steel frame structures during the 1906 San Francisco earthquake most likely stems from the capacity of the masonry wall to carry the lateral load demands of the structure. Because the masonry takes those loads until collapse, the steel frame itself experiences very little significant damage, even under severe loading conditions, an effect which is reflected in the performance of the prototypical building under the highest target displacement demands. While building hazards after a severe earthquake are highest in those highest seismic risk zones, even regions of comparatively low seismic risk have a likelihood of significant visible damage to masonry panels of an infill-frame building. It is important for engineers in these regions to be aware that if a major earthquake occurs, life safety hazards will occur within those infill components.

Finally, while expected damage would be as described for a structure similar to the prototypical building in any of the seventeen cities studied, those expected types of damage also relate specifically to the parameters used for this project. For example, if a building of similar construction to the prototypical building exists in poor soil conditions, as discussed in section 5.0 in relation to San Francisco and Chicago, this situation would increase the predicted target displacement, an effect which would correspond to a higher extent of damage within the structure. Other factors which may increase the amount of

damage expected in an individual structure of this construction (steel frame with URM infill) include previous damage already present in the structure and original construction quality – especially in relation to the interaction between the steel frame and URM infill.

It is critical for engineers to be aware that recent code adaptations and changes in accepted guidelines increase the expected lateral load resisting requirements in most seismic risk regions. These changes correspond with increased target displacement demands for existing structures like those studied in this project, especially in lower seismic risk zones. While structures similar to the prototypical building in a low risk zone – like those locations categorized as Group 1 or Group 2 cities in this study – during a severe earthquake may be able to resist a majority of the applied seismic loads, the forces experienced may still result in significant damage. The implication of this comparison between target displacement demands and the pushover response of the prototypical building is that even in regions with little historic record of significant earthquakes, engineers need to be aware of the possibility for specific types of damage to occur so they can take appropriate measures to strengthen an existing building in anticipation of a major earthquake.

6.2 Recommendations for Future Research

In a review of the work completed for this project and associated conclusions, areas for future research become apparent. The use, behavior, and performance of infill wall panels – including steel frames with URM infill walls – historically and in new construction is still not fully understood, and there are still advances in the existing body

of knowledge and rehabilitation guidelines to make through continued investigation of existing structures, physical testing, and computer modeling.

Within existing steel construction with URM infill, it is important that as buildings are modified or retrofitted, engineers continue to thoroughly evaluate the structure in order to better understand this system's performance over time and to better understand regional construction variation. Documentation of actual building performance and construction can help with the continued development of testing and modeling procedures.

Although there is a lack of testing on existing buildings due to access difficulty, there have been a multitude of tests completed by universities and professional engineers to understand the in- and out-of-plane behavior of infill walls and the interaction between the infill panel and surrounding frame. However, it appears that a majority of these tests, especially within the last twenty years, have been performed on test subjects with either HCT (hollow clay tile) or concrete infill and often with concrete frames. Therefore, a possible area of further study is the testing of the particular structural system focused on in this project (steel frame construction with URM infill panels). It would be especially interesting if test subjects used historic steel construction methods for connections (riveting) and cross-sectional geometry and historic brick and mortar make-up as opposed to modern methods and materials of construction. The replication of historic construction for testing purposes would be an extremely useful learning tool for engineers working with early steel -frame buildings.

Additionally, another area of study with a lack of testing data seems to be infill panels with perforations: because most steel frame buildings with URM infill dating to the late nineteenth and early twentieth centuries have at least one elevation with many openings for windows and doors, it would be of great use to better understand how perforated frame-infill configurations perform. Along with greater knowledge of the performance of perforated infill components, a well defined process for modeling the behavior of those components, as opposed to adapting the existing methods for solid panels to a panel with openings, would help in behavior predictions at a structural level.

Finally, the ability to interpret and apply existing rehabilitation standards to an individual project is critical to many structural engineering projects. For infill wall construction, the currently accepted ASCE 41 guidelines are minimal, especially in comparison to other material or component sections of the guidelines. For example, in nonlinear analyses, the guideline for masonry infill walls ignores stiffness degradation and residual strength and only gives a performance level relationship to the yield point for the Life Safety performance level. Consequently, the nonlinear behavior of computer modeled structural components is constant after initial yielding and there is a lack of information for design at other performance levels.

Understanding the definitions given by ASCE 41 for infill wall construction is important because direction given with respect to nonlinear modeling requires the engineer following ASCE 41 to perform a finite element analysis of the interaction between the frame and infill panel, something which may be impractical for an actual project. Currently, software programs used by many structural engineering firms to

model structures for nonlinear behavior, including ETABS and PERFORM, do not have the modeling capability to determine a full backbone curve for infill materials as is possible with a standard wall or column section. Also, while ASCE 41 explains how a perforated panel differs from a solid infill panel, it does not give guidance for the computer modeling of a perforated panel, adding to the conservative nature of a computer model following ASCE 41 guidelines.

Additional computer modeling of steel-frame structures with URM infill is important to improving the guidelines in ASCE 41 and the improvement of existing computer analysis programs. Continuing studies of prototypical buildings, like the structure created for this project, are important for better understanding how a specific structural system performs in different seismic risk regions and for continuing to understand the affect of new building code adaptations to new and existing structural design. Steel frame construction and performance in the late nineteenth and early twentieth centuries was critical to the development of modern steel construction methods. Continued research into the design, behavior, and performance of those early steel frame systems will help current engineers to better understand steel construction and to approach rehabilitation projects of early steel frame buildings.

7.0 LIST OF TERMS

CMU Concrete masonry unit

HCT Hollow clay tile

URM Unreinforced masonry

AISC American Institute of Steel Construction

ASCE American Society of Civil Engineers

ATC Applied Technology Council

FEMA Federal Emergency Management Agency

USGS United States Geologic Survey

ETABS An analysis program for modeling linear and nonlinear structural behavior

PERFORM An analysis program for modeling nonlinear structural behavior

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APPENDICES

APPENDIX A: PERFORMANCE IN VARYING SEISMIC RISK REGIONS

GROUP 1

Geographic Locations:

- Regions of little to no seismic activity
- Peak Ground Acceleration for a 2% probability of exceedance in 50 years ranges from 0% to 8%
- Less than 0.28% drift
- Includes Baltimore, Chicago, Cleveland, Detroit, Minneapolis, and Washington, DC

Figure 1: Group 1 Cities

Source: Author

Life Safety:

When expected target

displacements are reached for cities in Group 1, existing structures of this type meet most Life Safety requirements defined in ASCE 41.

Deficiencies in meeting these requirements are:

- Possible collapse of URM bricks in 1st mezzanine – 5th stories

Extent of Damage

Possible damage to existing structures in Group 1 regions include:

Infill Walls

- Cracking of masonry construction throughout structure, but most severe cracking concentrated in 1st mezzanine – 5th stories
- Possibility of falling bricks, especially at stiffest levels of 1st mezzanine – 5th stories

Steel Frame

- No significant damage to the steel frame

Figure 2: Structural Damage Areas

Source: Author

Seismic Strengthening

Some strengthening may be required for infill walls in 1st mezzanine – 5th stories in order to avoid severe damage to masonry construction.

GROUP 2

Geographic Locations:

- Regions of low seismic activity
- Peak Ground Acceleration for a 2% probability of exceedance in 50 years ranges from 8% to 16%
- 0.28% to 0.44% drift
- Includes Atlanta, Birmingham, Boston, and New York

Figure 3: Group 2 Cities

Source: Author

Life Safety:

When expected target displacements are reached for cities in Group 2, existing structures of this type meet most Life Safety requirements defined in ASCE 41.

Deficiencies in meeting these requirements are:

- Possible collapse of URM bricks and URM wall panels in 1st mezzanine – 5th stories

Extent of Damage

Possible damage to existing structures in Group 2 regions include:

Infill Walls

- Cracking of masonry construction throughout structure, but most severe cracking concentrated in 1st mezzanine – 5th stories
- Possibility of falling bricks, especially in 1st mezzanine – 5th stories
- Possibility of falling panels, especially at stiffest levels 1st mezzanine – 5th stories

Steel Frame

- No significant damage to the steel frame

Figure 4: Structural Damage Areas

Source: Author

Seismic Strengthening

Strengthening is required for infill walls, particularly in the lower half of the structure in order to avoid severe damage to masonry construction.

GROUP 3

Geographic Locations:

- Regions of moderate seismic activity
- Peak Ground Acceleration for a 2% probability of exceedance in 50 years ranges from 16% to 32%
- 0.44% to 0.67% drift
- Includes Nashville and St. Louis

Figure 5: Group 3 Cities

Source: Author

Life Safety:

When expected target displacements are reached for cities in Group 3, existing structures of this type do not always meet Life Safety requirements defined in ASCE 41.

Deficiencies in meeting these requirements are:

- Possible collapse of URM bricks and URM wall panels in 1st mezzanine – 6th stories

Extent of Damage

Possible damage to existing structures in Group 3 regions include:

Infill Walls

- Severe cracking of masonry construction throughout structure
- Possibility of falling bricks, except at the upper stories of the building
- Possibility of falling panels in 1st mezzanine – 6th stories

Steel Frame

- No significant damage to the steel frame

Figure 6: Structural Damage Areas

Source: Author

Seismic Strengthening

Strengthening is required for infill walls, particularly in 1st mezzanine – 6th stories in order to avoid severe damage to masonry construction.

GROUP 4

Geographic Locations:

- Regions of high seismic activity
- Peak Ground Acceleration for a 2% probability of exceedance in 50 years ranges from 32% to 64%
- 0.94% to 1.39% drift
- Includes Charleston, Seattle, and Portland

Figure 7: Group 4 Cities

Source: Author

Life Safety:

When expected target displacements are reached for cities in Group 4, existing structures of this type do not meet all Life Safety requirements defined in ASCE 41.

Deficiencies in meeting these requirements are:

- Collapse of URM bricks and URM wall panels in 1st mezzanine – 7th stories

Extent of Damage

Possible damage to existing structures in Group 4 regions include:

Infill Walls

- Cracking of masonry construction throughout structure
- Possibility of falling bricks and falling panels in 1st mezzanine – 7th stories

Steel Frame

- Visible cracking of confining concrete and paint flaking throughout 1st mezzanine – 6th stories
- Possible prying of connections throughout 1st mezzanine – 6th stories

Figure 8: Structural Damage Areas

Source: Author

Seismic Strengthening

Strengthening is required for infill walls throughout the structure in order to avoid severe damage to masonry construction. Depending on performance goals, strengthening may also be required in the steel frame to prevent significant connection damage.

GROUP 5

Geographic Locations:

- Regions of very high seismic activity
- Peak Ground Acceleration for a 2% probability of exceedance in 50 years is more than 64%
- 1.67% to 2.22% drift
- Includes Los Angeles and San Francisco

Figure 9: Group 5 Cities

Source: Author

Life Safety:

When expected target displacements are reached for cities in Group 5, existing structures of this type do not meet all Life Safety requirements defined in ASCE 41.

Deficiencies in meeting these requirements are:

- Collapse of URM bricks and URM wall panels throughout the building
- Possible beam component buckling or connection failure in 1st mezzanine – 5th stories

Extent of Damage

Possible damage to existing structures in Group 5 regions include:

Infill Walls

- Severe cracking of masonry construction throughout structure
- Possibility of falling bricks and falling panels throughout structure

Steel Frame

- Cracked confining concrete paint flaking, and prying of connections throughout structure
- Beam buckling, distortion and connection failure in 1st mezzanine – 5th stories

Figure 10: Structural Damage Areas

Source: Author

Seismic Strengthening

Strengthening is required for infill walls throughout the structure in order to avoid severe damage to masonry construction. Additional analysis is required to better predict behavior of infill components. Strengthening will also be required in the steel frame to prevent critical beam damage.

APPENDIX B: COMPUTER MODEL DOCUMENTATION AND VERIFICATION

Computer model documentation which began in Section 4.0 follows.

Figure 11: Calibration Model Elevation
Source: Author, using ETABS

Figure 12: Calibration Model Hinge Locations
Source: Author, using ETABS

Material Property Data

Material Name STEEL

Display Color Color [Red]

Type of Material
☒ Isotropic ☐ Orthotropic

Type of Design
 Design Steel

Analysis Property Data

Mass per unit Volume	0.
Weight per unit Volume	0.
Modulus of Elasticity	29000.
Poisson's Ratio	0.3
Coeff of Thermal Expansion	0.
Shear Modulus	11153.8462

Design Property Data

Minimum Yield Stress, Fy	36.
Minimum Tensile Strength, Fu	58.
Cost per Unit Weight	1.

OK Cancel

Figure 15: Calibration Model Steel Material Definition

Source: Author, using ETABS

I/Wide Flange Section

Section Name W12X35

Extract Data from Section Property File
 Open File... C:\Program Files\Computers and Import...

Properties Section Properties... **Property Modifiers** Set Modifiers... **Material** STEEL

Dimensions

Outside height (t3)	12.5
Top flange width (t2)	6.56
Top flange thickness (tf)	0.52
Web thickness (tw)	0.3
Bottom flange width (t2b)	6.56
Bottom flange thickness (tfb)	0.52

Display Color [Red]

OK

Figure 16: Calibration Model Beam Cross-Section

Source: Author, using ETABS

Figure 17: Calibration Model Column Cross-Section
Source: Author, using ETABS

Figure 18: Calibration Model Strut Cross-Section
Source: Author, using ETABS

Frame Hinge Property Data for BEAM - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-100
D-	-1	-100
C-	-1	-100
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	100.
D	1.	100.
E	1.	100.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☒ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	2.	
Life Safety	4.	
Collapse Prevention	6.	

OK
Cancel

Figure 19: Calibration Model Beam Nonlinear Hinge Definition
Source: Author, using ETABS

Frame Hinge Property Data for COLUMN - PM1

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-100
D-	-1	-100
C-	-1	-100
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	100.
D	1.	100.
E	1.	100.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☐ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	2.	
Life Safety	4.	
Collapse Prevention	6.	

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation
☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 20: Calibration Model Column Nonlinear Hinge Definition
Source: Author, using ETABS

Frame Hinge Property Data for STRUT - P

Edit

Point	Force/SF	Disp/SF
E-	-1	-100
D-	-1	-100
C-	-1	-100
B-	-1	0
A	0	0
B	1	0
C	1	100
D	1	100
E	1	100

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Force and Disp

	Positive	Negative
☒ Use Yield Force	Force SF	
☒ Use Yield Disp	Disp SF	

Acceptance Criteria (Plastic Disp/SF)

	Positive	Negative
Immediate Occupancy	2	
Life Safety	4	
Collapse Prevention	6	

Type

☒ Force - Displacement

☐ Stress - Strain

Hinge Length

☐ Relative Length

OK
Cancel

Figure 21: Calibration Model Strut Nonlinear Hinge Definition
Source: Author, using ETABS

Figure 22: Calibration Model Nonlinear Pushover Output
Source: Author, using ETABS

PUSHOVER CURVE

File

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0000	0.0000	6	1	0	0	0	0	0	0	7
1	0.2814	33.2345	6	0	1	0	0	0	0	0	7
2	1.2814	38.2590	5	1	0	0	1	0	0	0	7
3	2.0039	41.8895	4	2	0	0	1	0	0	0	7
4	2.0343	41.9766	4	0	2	0	1	0	0	0	7
5	3.0343	42.5016	4	0	0	0	3	0	0	0	7
6	4.0343	43.0266	4	0	0	0	3	0	0	0	7
7	5.0343	43.5516	4	0	0	0	3	0	0	0	7
8	6.0343	44.0767	4	0	0	0	3	0	0	0	7
9	7.0343	44.6017	4	0	0	0	3	0	0	0	7
10	8.0343	45.1267	3	1	0	0	3	0	0	0	7
11	8.8951	45.5787	3	1	0	0	3	0	0	0	7
12	9.8951	45.8913	3	1	0	0	3	0	0	0	7
13	10.0000	45.9241	7	0	0	0	0	0	0	0	7

Figure 23: Calibration Model Nonlinear Pushover Data
Source: Author, using ETABS

Define Grid Data

Edit Format

X Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.	Grid Color
1	A	0.	Primary	Show	Top	
2	B	17.8333	Primary	Show	Top	
3	C	37.6667	Primary	Show	Top	
4	D	57.5	Primary	Show	Top	
5	E	68.1667	Primary	Show	Top	
6	F	84.8333	Primary	Show	Top	
7	G	95.5	Primary	Show	Top	
8	H	115.3333	Primary	Show	Top	
9	I	135.1667	Primary	Show	Top	
10	J	153.	Primary	Show	Top	

Y Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.	Grid Color
1	1	0.	Primary	Show	Left	
2	2	21.25	Primary	Show	Left	
3	3	41.0833	Primary	Show	Left	
4	4	60.9167	Primary	Show	Left	
5	5	80.75	Primary	Show	Left	
6	6	100.5833	Primary	Show	Left	
7	7	120.4167	Primary	Show	Left	
8	8	141.6667	Primary	Show	Left	
9						
10						

Units: Kip-ft

Display Grids as:
☒ Ordinates ☐ Spacing

☐ Hide All Grid Lines
☐ Glue to Grid Lines

Bubble Size: 5

Reset to Default Color
 Reorder Ordinates

OK Cancel

Figure 24: Plan Dimension Input
 Source: Author, using ETABS

Story Data

	Label	Height	Elevation	Master Story	Similar To	Splice Point	Splice Height
13	ROOF	12.5833	147.6667	No	NONE	No	0.
12	8TH	10.	135.0833	No	NONE	No	0.
11	7TH	10.	125.0833	Yes		No	0.
10	6TH	16.25	115.0833	No	NONE	No	0.
9	5THMEZ	12.25	98.8333	No	NONE	No	0.
8	5TH	13.6667	86.5833	No	NONE	No	0.
7	4THMEZ	10.6667	72.9167	No	NONE	No	0.
6	4TH	12.5	62.25	No	NONE	No	0.
5	3RD	12.6667	49.75	No	NONE	No	0.
4	2NDMEZ	13.8333	37.0833	No	NONE	No	0.
3	2ND	10.	23.25	No	NONE	No	0.
2	1STMEZ	13.25	13.25	No	NONE	No	0.
1	BASE		0.				

Reset Selected Rows

Height: 12.5833 Reset
 Master Story: No Reset
 Similar To: NONE Reset
 Splice Point: No Reset
 Splice Height: 0. Reset

Units: Change Units: Kip-ft

OK Cancel

Figure 25: Story Height Input
 Source: Author, using ETABS

Figure 26: First-Story Mezzanine Plan
Source: Author, using ETABS

Figure 27: Second-Story Plan
Source: Author, using ETABS

Figure 28: Second-Story Mezzanine Plan
Source: Author, using ETABS

Figure 29: Third-Story Plan
Source: Author, using ETABS

Figure 30: Fourth-Story Plan
Source: Author, using ETABS

Figure 31: Fourth-Story Mezzanine Plan
Source: Author, using ETABS

Figure 32: Fifth-Story Plan
Source: Author, using ETABS

Figure 33: Fifth-Story Mezzanine Plan
Source: Author, using ETABS

Figure 34: Sixth-Story Plan
Source: Author, using ETABS

Figure 35: Seventh-Story Plan
Source: Author, using ETABS

Figure 36: Eighth-Story Plan
Source: Author, using ETABS

Figure 37: Roof Plan
Source: Author, using ETABS

Figure 38: Grid Line 1 Elevation
Source: Author, using ETABS

Figure 39: Grid Line 2 Elevation
Source: Author, using ETABS

Figure 40: Grid Line 3 Elevation
Source: Author, using ETABS

Figure 41: Grid Line 4 Elevation
Source: Author, using ETABS

Figure 42: Grid Line 5 Elevation
Source: Author, using ETABS

Figure 43: Grid Line 6 Elevation
Source: Author, using ETABS

Figure 44: Grid Line 7 Elevation
Source: Author, using ETABS

Figure 45: Grid Line 8 Elevation
Source: Author, using ETABS

Figure 46: Grid Line A Elevation
Source: Author, using ETABS

Figure 47: Grid Line B Elevation
Source: Author, using ETABS

Figure 48: Grid Line C Elevation
Source: Author, using ETABS

Figure 49: Grid Line D Elevation
Source: Author, using ETABS

Figure 50: Grid Line E Elevation
Source: Author, using ETABS

Figure 51: Grid Line F Elevation
Source: Author, using ETABS

Figure 52: Grid Line G Elevation

Source: Author, using ETABS

Figure 53: Grid Line H Elevation
Source: Author, using ETABS

Figure 54: Grid Line I Elevation
Source: Author, using ETABS

Figure 55: Grid Line J Elevation
Source: Author, using ETABS

Figure 56: 3-D View, Front and Side
Source: Author, using ETABS

Figure 57: 3-D View, Back
Source: Author, using ETABS

Figure 58: 3-D View, Back and Side
Source: Author, using ETABS

Figure 59: 3-D View, Top
Source: Author, using ETABS

Material Property Data

Material Name CONC

Type of Material
☒ Isotropic ☐ Orthotropic

Analysis Property Data
 Mass per unit Volume 0.
 Weight per unit Volume 0.
 Modulus of Elasticity 3600.
 Poisson's Ratio 0.2
 Coeff of Thermal Expansion 0.
 Shear Modulus 1500.

Display Color
 Color [Cyan Box]

Type of Design
 Design Concrete

Design Property Data (ACI 318-99)
 Specified Conc Comp Strength, f'c 4.
 Bending Reinf. Yield Stress, fy 60.
 Shear Reinf. Yield Stress, fys 60.
☐ Lightweight Concrete
 Shear Strength Reduc. Factor

OK Cancel

Figure 60: Concrete Property Input (k-in)
 Source: Author, using ETABS

Material Property Data

Material Name MASONRY

Type of Material
☒ Isotropic ☐ Orthotropic

Analysis Property Data
 Mass per unit Volume 0.
 Weight per unit Volume 0.
 Modulus of Elasticity 330.
 Poisson's Ratio 0.15
 Coeff of Thermal Expansion 0.
 Shear Modulus 143.4783

Display Color
 Color [Green Box]

Type of Design
 Design Concrete

Design Property Data (ACI 318-99)
 Specified Conc Comp Strength, f'c 0.6
 Bending Reinf. Yield Stress, fy 0.
 Shear Reinf. Yield Stress, fys 0.
☐ Lightweight Concrete
 Shear Strength Reduc. Factor

OK Cancel

Figure 61: Masonry Property Input (k-in)
 Source: Author, using ETABS

Material Property Data

Material Name: STEEL

Display Color:

Type of Material: ☒ Isotropic ☐ Orthotropic

Type of Design: Design: Steel

Analysis Property Data:

Mass per unit Volume	0.
Weight per unit Volume	0.
Modulus of Elasticity	29000.
Poisson's Ratio	0.3
Coeff of Thermal Expansion	0.
Shear Modulus	11153.8462

Design Property Data:

Minimum Yield Stress, Fy	30.
Minimum Tensile Strength, Fu	60.
Cost per Unit Weight	1.

OK Cancel

Figure 62: Steel Property Input (k-in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name: 12I31.5

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions:

Outside height (t3)	12.
Top flange width (t2)	5.
Top flange thickness (tf)	0.544
Web thickness (tw)	0.35
Bottom flange width (t2b)	5.
Bottom flange thickness (tfb)	0.544

Display Color:

OK Cancel

Figure 63: 12I31.5 Beam Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name 18I55

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	18.
Top flange width (t2)	6.
Top flange thickness (tf)	0.69
Web thickness (tw)	0.46
Bottom flange width (t2b)	6.
Bottom flange thickness (tfb)	0.69

Display Color: ■

OK Cancel

Figure 64: 18I55 Beam Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name 20I65

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	15.
Top flange width (t2)	6.098
Top flange thickness (tf)	0.815
Web thickness (tw)	0.688
Bottom flange width (t2b)	6.098
Bottom flange thickness (tfb)	0.815

Display Color: ■

OK Cancel

Figure 65: 20I65 Beam Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C1-14x5/8x6x3/4

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	14.
Top flange width (t2)	12.625
Top flange thickness (tf)	0.75
Web thickness (tw)	0.625
Bottom flange width (t2b)	12.625
Bottom flange thickness (tfb)	0.75

Display Color ☒

OK Cancel

Figure 66: First-Story and First-Story Mezzanine Column Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C2-14x5/8x6x4x3/4

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	14.
Top flange width (t2)	8.625
Top flange thickness (tf)	0.75
Web thickness (tw)	0.625
Bottom flange width (t2b)	8.625
Bottom flange thickness (tfb)	0.75

Display Color ☐

OK Cancel

Figure 67: Second-Story and Second-Story Mezzanine Column Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C3-12x5/8x6x4x3/4

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	12.
Top flange width (t2)	8.625
Top flange thickness (tf)	0.625
Web thickness (tw)	0.75
Bottom flange width (t2b)	8.625
Bottom flange thickness (tfb)	0.625

Display Color: ■

OK Cancel

Figure 68: Third-Story Column Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C4-12x5/8x6x4x3/4

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	12.
Top flange width (t2)	8.625
Top flange thickness (tf)	0.75
Web thickness (tw)	0.625
Bottom flange width (t2b)	8.625
Bottom flange thickness (tfb)	0.75

Display Color: ■

OK Cancel

Figure 69: Fourth-Story and Fourth-Story Mezzanine Column Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C5-12x3/8x6x4x3/8

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	12.
Top flange width (t2)	8.375
Top flange thickness (tf)	0.375
Web thickness (tw)	0.375
Bottom flange width (t2b)	8.375
Bottom flange thickness (tfb)	0.375

Diagram: I-section with dimensions 2 and 3 indicated. Display Color: ■

OK Cancel

Figure 70: Fifth-Story and Fifth-Story Mezzanine Column Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name C6-12x1/2x6x4x3/4

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3)	12.
Top flange width (t2)	8.25
Top flange thickness (tf)	0.75
Web thickness (tw)	0.5
Bottom flange width (t2b)	8.25
Bottom flange thickness (tfb)	0.75

Diagram: I-section with dimensions 2 and 3 indicated. Display Color: ■

OK Cancel

Figure 71: Sixth-Story Column Input (in)
Source: Author, using ETABS

The screenshot shows the 'I/Wide Flange Section' dialog box. The 'Section Name' field contains 'C7-12x5/16x4x3x5/16'. The 'Material' dropdown is set to 'STEEL'. The 'Dimensions' section includes the following values: Outside height (t3) = 12, Top flange width (t2) = 6.3125, Top flange thickness (tf) = 0.3125, Web thickness (tw) = 0.3125, Bottom flange width (t2b) = 6.3125, and Bottom flange thickness (tfb) = 0.3125. A preview diagram on the right shows an I-section with dimensions 2 and 3 indicated. The 'Display Color' checkbox is unchecked. The 'OK' and 'Cancel' buttons are at the bottom.

Figure 72: Seventh-Story and Eighth-Story Column Input (in)
Source: Author, using ETABS

The screenshot shows the 'I/Wide Flange Section' dialog box for a transfer girder. The 'Section Name' field contains 'TRANSFERGIRDER1'. The 'Material' dropdown is set to 'STEEL'. The 'Dimensions' section includes the following values: Outside height (t3) = 52, Top flange width (t2) = 18, Top flange thickness (tf) = 0.75, Web thickness (tw) = 1, Bottom flange width (t2b) = 18, and Bottom flange thickness (tfb) = 0.75. A preview diagram on the right shows an I-section with dimensions 2 and 3 indicated. The 'Display Color' checkbox is checked, and a color swatch is visible. The 'OK' and 'Cancel' buttons are at the bottom.

Figure 73: Transfer Girder 1 Input (in)
Source: Author, using ETABS

I/Wide Flange Section

Section Name TRANSFERGIRDER2

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: STEEL

Dimensions

Outside height (t3) 52.
 Top flange width (t2) 18.
 Top flange thickness (tf) 0.75
 Web thickness (tw) 1.
 Bottom flange width (t2b) 18.
 Bottom flange thickness (tfb) 0.75

Display Color ☐

OK Cancel

Figure 74: Transfer Girder 2 Column Input (in)
 Source: Author, using ETABS

Rectangular Section

Section Name S1-1-1

Properties: Section Properties... Property Modifiers: Set Modifiers... Material: MASONRY

Dimensions

Depth (t3) 16.
 Width (t2) 17.

Concrete: Reinforcement...

Display Color ☒

OK Cancel

Figure 75: Example Equivalent Strut Input (in)
 Source: Author, using ETABS

**Diagonal Compression Strut Calculations:
Sample Calculation**

Material and Geometric Properties of URM Infill

h_{inf}	151	in	f'_m	600	psi
L_{inf}	200	in	f_{vie}	20	psi
t_{inf}	13	in			

$\theta_{Equiv. \text{ Strut}}$	0.646693	radians
r_{inf}	250.60	in

Material and Geometric Properties of Steel Frame

h_{Col}	120	in	E_{fc}	29000	ksi
I_{Col}	50.25	in ⁴	F_{ye}	30	ksi

Stiffness Calculations

E_{mc}	330	ksi	(Eq. 7-7)
λ_1	0.04653		
a	22.04	in	
A	286.55	in ²	
$k_{Elastic}$	240.34	k/in	

Strength Calculations

A_{ni}	2600	in ²	(Eq. 7-8)
V_{inf}	52.00	k	
Strut Load	41.50019	k	

Force-Displacement Relationship

Δ_y	0.216	in
V_{fre}	35.50781	k
β	0.68	
L_{inf}/h_{inf}	1.324503

The image shows a software dialog box titled "Wall/Slab Section". It contains several input fields and options for defining a structural section. The "Section Name" field is set to "DIAPHRAGM". The "Material" dropdown menu is set to "CONC". Under the "Thickness" section, both "Membrane" and "Bending" are set to "5". In the "Type" section, the "Shell" radio button is selected, and the "Thick Plate" checkbox is unchecked. In the "Load Distribution" section, the "Use Special One-Way Load Distribution" checkbox is unchecked. At the bottom, there is a "Set Modifiers..." button, a "Display Color" option with a magenta color swatch, and "OK" and "Cancel" buttons.

Field/Option	Value/Setting
Section Name	DIAPHRAGM
Material	CONC
Thickness - Membrane	5
Thickness - Bending	5
Type - Shell	Selected
Type - Membrane	Not Selected
Type - Plate	Not Selected
Type - Thick Plate	Not Selected
Load Distribution - Use Special One-Way Load Distribution	Not Selected
Display Color	Magenta

Figure 76: Shell Element Input (in)
Source: Author, using ETABS

Figure 77: Equivalent Strut End Releases, Grid Line 1
Source: Author, using ETABS

Figure 78: Equivalent Strut End Releases, Grid Line 3
Source: Author, using ETABS

Figure 79: Equivalent Strut End Releases, Grid Line 5
Source: Author, using ETABS

Figure 80: Equivalent Strut End Releases, Grid Line 8
Source: Author, using ETABS

Figure 81: Equivalent Strut End Releases, Grid Line A
Source: Author, using ETABS

Figure 82: Equivalent Strut End Releases, Grid Line D
Source: Author, using ETABS

Figure 83: Equivalent Strut End Releases, Grid Line G
Source: Author, using ETABS

Figure 84: Equivalent Strut End Releases, Grid Line J
Source: Author, using ETABS

Figure 85: First-Story Mezzanine Diaphragm Extent
Source: Author, using ETABS

Figure 86: Second-Story Diaphragm Extent
Source: Author, using ETABS

Figure 87: Second-Story Mezzanine Diaphragm Extent
Source: Author, using ETABS

Figure 88: Third-Story Diaphragm Extent
Source: Author, using ETABS

Figure 89: Fourth-Story Diaphragm Extent
Source: Author, using ETABS

Figure 90: Fourth-Story Mezzanine Diaphragm Extent
Source: Author, using ETABS

Figure 91: Fifth-Story Diaphragm Extent
Source: Author, using ETABS

Figure 92: Fifth-Story Mezzanine Diaphragm Extent
Source: Author, using ETABS

Figure 93: Sixth-Story Diaphragm Extent
Source: Author, using ETABS

Figure 94: Seventh-Story Diaphragm Extent
Source: Author, using ETABS

Figure 95: Eighth-Story Diaphragm Extent
Source: Author, using ETABS

Figure 96: Roof Diaphragm Extent
Source: Author, using ETABS

Figure 97: First-Story Mezzanine Line and Area Masses
Source: Author, using ETABS

Figure 98: Second-Story Area and Line Masses
Source: Author, using ETABS

Figure 99: Second-Story Mezzanine Area and Line Masses
Source: Author, using ETABS

Figure 100: Third-Story Area and Line Masses
Source: Author, using ETABS

Figure 101: Fourth-Story Area and Line Masses
Source: Author, using ETABS

Figure 102: Fourth-Story Mezzanine Area and Line Masses
Source: Author, using ETABS

Figure 103: Fifth-Story Area and Line Masses
Source: Author, using ETABS

Figure 104: Fifth-Story Mezzanine Area and Line Masses
Source: Author, using ETABS

Figure 105: Sixth-Story Area and Line Masses
Source: Author, using ETABS

Figure 106: Seventh-Story Area and Line Masses
Source: Author, using ETABS

Figure 107: Eighth-Story Area and Line Masses
Source: Author, using ETABS

Figure 108: Roof Area and Line Masses
Source: Author, using ETABS

Define Static Load Case Names

Load	Type	Self Weight Multiplier	Auto Lateral Load
DEAD	DEAD	0	
DEAD	DEAD	0	
LIVE	LIVE	0	
POINT	QUAKE	0	User Loads
POINTY	QUAKE	0	User Loads

Click To:

Add New Load

Modify Load

Modify Lateral Load...

Delete Load

OK

Cancel

Figure 109: Static Load Definitions

Source: Author, using ETABS

Figure 110: First-Story Mezzanine Area Dead Loads
Source: Author, using ETABS

Figure 111: Second-Story Area Dead Loads
Source: Author, using ETABS

Figure 112: Second-Story Mezzanine Area Dead Loads
Source: Author, using ETABS

Figure 113: Third-Story Area Dead Loads
Source: Author, using ETABS

Figure 114: Fourth-Story Area Dead Loads
Source: Author, using ETABS

Figure 115: Fourth-Story Mezzanine Area Dead Loads
Source: Author, using ETABS

Figure 116: Fifth-Story Area Dead Loads
Source: Author, using ETABS

Figure 117: Fifth-Story Mezzanine Area Dead Loads
Source: Author, using ETABS

Figure 118: Sixth-Story Area Dead Loads
Source: Author, using ETABS

Figure 119: Seventh-Story Area Dead Loads
Source: Author, using ETABS

Figure 120: Eighth-Story Area Dead Loads
Source: Author, using ETABS

Figure 121: Roof Area Dead Loads
Source: Author, using ETABS

Figure 122: Grid Line 1 Dead Loads
Source: Author, using ETABS

Figure 123: Grid Line 3 Dead Loads
Source: Author, using ETABS

Figure 124: Grid Line 5 Dead Loads
Source: Author, using ETABS

Figure 125: Grid Line 8 Dead Loads
Source: Author, using ETABS

Figure 126: Grid Line A Dead Loads
Source: Author, using ETABS

Figure 127: Grid Line D Dead Loads
Source: Author, using ETABS

Figure 128: Grid Line G Dead Loads
Source: Author, using ETABS

Figure 129: Grid Line J Dead Loads
Source: Author, using ETABS

Figure 130: First-Story Mezzanine Area Live Loads
Source: Author, using ETABS

Figure 131: Second-Story Area Live Loads
Source: Author, using ETABS

Figure 132: Second-Story Mezzanine Area Live Loads
Source: Author, using ETABS

Figure 133: Third-Story Area Live Loads
Source: Author, using ETABS

Figure 134: Fourth-Story Area Live Loads
Source: Author, using ETABS

Figure 135: Fourth-Story Mezzanine Area Live Loads
Source: Author, using ETABS

Figure 136: Fifth-Story Area Live Loads
Source: Author, using ETABS

Figure 137: Fifth-Story Mezzanine Area Live Loads

Source: Author, using ETABS

Figure 138: Sixth-Story Area Live Loads
Source: Author, using ETABS

Figure 139: Seventh-Story Area Live Loads
Source: Author, using ETABS

Figure 140: Eighth-Story Area Live Loads
Source: Author, using ETABS

Figure 141: Roof Area Live Loads
Source: Author, using ETABS

User Seismic Loading
Edit

User Seismic Loads on Diaphragms

Story	Diaphragm	FX	FY	MZ
ROOF	D1	10000.	0.	0.
8TH	D1	9147.9	0.	0.
7TH	D1	8470.7	0.	0.
6TH	D1	7793.5	0.	0.
5THMEZ	D1	6693.	0.	0.
5TH	D1	5863.4	0.	0.
4THMEZ	D1	4937.9	0.	0.
4TH	D1	4215.6	0.	0.
3RD	D1	3369.1	0.	0.
2NDMEZ	D1	2511.3	0.	0.
2ND	D1	1574.5	0.	0.
1STMEZ	D1	897.3	0.	0.

☐ User Specified Application Point
☒ Apply at Center of Mass

Additional Ecc. Ratio (all Diaph.)

OK Cancel

Figure 142: Lateral Load Definition, X-Direction
Source: Author, using ETABS

User Seismic Loading
Edit

User Seismic Loads on Diaphragms

Story	Diaphragm	FX	FY	MZ
ROOF	D1	0.	10000.	0.
8TH	D1	0.	9147.9	0.
7TH	D1	0.	8470.7	0.
6TH	D1	0.	7793.5	0.
5THMEZ	D1	0.	6693.	0.
5TH	D1	0.	5863.4	0.
4THMEZ	D1	0.	4937.9	0.
4TH	D1	0.	4215.6	0.
3RD	D1	0.	3369.1	0.
2NDMEZ	D1	0.	2511.3	0.
2ND	D1	0.	1574.5	0.
1STMEZ	D1	0.	897.3	0.

☐ User Specified Application Point
☒ Apply at Center of Mass

Additional Ecc. Ratio (all Diaph.)

OK Cancel

Figure 143: Lateral Load Definition, Y-Direction
Source: Author, using ETABS

Static Nonlinear Case Data

Static Nonlinear Case Name: DEAD

Options:

- ☒ Load to Level Defined by Pattern
- ☐ Push to Disp. Magnitude
- ☐ Use Conjugate Displ. for Control
- Monitor: UX, 1, ROOF
- Start from Previous Case: [Empty]
- ☒ Save Positive Increments Only
- Minimum Saved Steps: 1
- Maximum Null Steps: 50
- Maximum Total Steps: 200
- Maximum Iterations/Step: 10
- Iteration Tolerance: 1.000E-04
- Event Tolerance: 0.01

Member Unloading Method: Unload Entire Structure

Geometric Nonlinearity Effects: None

Load Pattern:

Load	Scale Factor
DEAD	1.
DEAD	1.

Buttons: Add, Modify, Delete

Active Structure:

Stage	Active Group
1	DEAD

Buttons: Add, Modify, Insert, Delete

☒ Loads Apply to Added Elements Only

OK Cancel

Figure 144: Nonlinear Static Load Definition, Dead Load
Source: Author, using ETABS

Static Nonlinear Case Data

Static Nonlinear Case Name: PUSH

Options:

- ☐ Load to Level Defined by Pattern
- ☒ Push to Disp. Magnitude: 100
- ☒ Use Conjugate Displ. for Control
- Monitor: UX, 1, ROOF
- Start from Previous Case: DEAD
- ☒ Save Positive Increments Only
- Minimum Saved Steps: 100
- Maximum Null Steps: 200
- Maximum Total Steps: 200
- Maximum Iterations/Step: 100
- Iteration Tolerance: 1.000E-04
- Event Tolerance: 0.01

Member Unloading Method: Restart Using Secant Stiffness

Geometric Nonlinearity Effects: P-Delta

Load Pattern:

Load	Scale Factor
POINT	1.
POINT	1.

Buttons: Add, Modify, Delete

Active Structure:

Stage	Active Group
1	ALL

Buttons: Add, Modify, Insert, Delete

☐ Loads Apply to Added Elements Only

OK Cancel

Figure 145: Nonlinear Static Load Definition, X-Direction
Source: Author, using ETABS

Static Nonlinear Case Data

Static Nonlinear Case Name:

Options

☐ Load to Level Defined by Pattern
☒ Push to Disp. Magnitude:
☒ Use Conjugate Displ. for Control
 Monitor:
 Start from Previous Case:
☒ Save Positive Increments Only

Minimum Saved Steps:
 Maximum Null Steps:
 Maximum Total Steps:
 Maximum Iterations/Step:
 Iteration Tolerance:
 Event Tolerance:

Member Unloading Method:

Geometric Nonlinearity Effects:

Load Pattern

Load	Scale Factor
POINTY	1.
POINTY	1.

Add Modify Delete

Active Structure

Stage	Active Group
1	ALL

Add Modify Insert Delete

☐ Loads Apply to Added Elements Only

OK Cancel

Figure 146: Nonlinear Static Load Definition, Y-Direction
 Source: Author, using ETABS

Frame Hinge Property Data for 12I315 - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☒ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	1.	
Life Safety	6.	
Collapse Prevention	8.	

OK
Cancel

Figure 147: 12I31.5 Beam Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for 18I55 - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☒ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	1.	
Life Safety	6.	
Collapse Prevention	8.	

OK
Cancel

Figure 148: 18I55 Beam Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for 20I65 - M3

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☒ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	1.	
Life Safety	6.	
Collapse Prevention	8.	

OK
Cancel

Figure 149: 20I65 Beam Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C1 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

	Positive	Negative
☐ Use Yield Moment	Moment SF	
☒ Use Yield Rotation	Rotation SF	

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	1.	
Life Safety	6.	
Collapse Prevention	8.	

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation
☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 150: First-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C1MEZ - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 1. 1.

Life Safety Positive Negative 6. 6.

Collapse Prevention Positive Negative 8. 8.

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation

☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 151: First-Story Mezzanine Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C2 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 1. 1.

Life Safety Positive Negative 6. 6.

Collapse Prevention Positive Negative 8. 8.

Axial Load - Displacement Relationship

☐ Proportional to Moment - Rotation

☒ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 152: Second-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C2MEZ - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 1.

Life Safety 6.

Collapse Prevention 8.

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation

☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 153: Second-Story Mezzanine Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C3 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 1.

Life Safety 6.

Collapse Prevention 8.

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation

☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 154: Third-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C4 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative

Life Safety Positive Negative

Collapse Prevention Positive Negative

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation

☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 155: Fourth-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C4MEZ - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative

Life Safety Positive Negative

Collapse Prevention Positive Negative

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation

☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 156: Fourth-Story Mezzanine Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C5 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 0.25 0.25

Life Safety Positive Negative 2. 2.

Collapse Prevention Positive Negative 3. 3.

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation
☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 157: Fifth-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C5MEZ - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 0.25 0.25

Life Safety Positive Negative 2. 2.

Collapse Prevention Positive Negative 3. 3.

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation
☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 158: Fifth-Story Mezzanine Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C6 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative
☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative
Life Safety Positive Negative
Collapse Prevention Positive Negative

Axial Load - Displacement Relationship

☐ Proportional to Moment - Rotation
☒ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 159: Sixth-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C7 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative
☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative
Life Safety Positive Negative
Collapse Prevention Positive Negative

Axial Load - Displacement Relationship

☒ Proportional to Moment - Rotation
☐ Elastic - Perfectly Plastic

Define/Show Interaction...

OK
Cancel

Figure 160: Seventh-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for C8 - PMM

Edit

Point	Moment/SF	Rotation/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Moment and Rotation

☐ Use Yield Moment Moment SF Positive Negative

☒ Use Yield Rotation Rotation SF Positive Negative

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy Positive Negative 0.25

Life Safety 2.

Collapse Prevention 3.

Axial Load - Displacement Relationship

☐ Proportional to Moment - Rotation

☒ Elastic - Perfectly Plastic

Define/Show Interaction...

OK

Cancel

Figure 161: Eighth-Story Column Nonlinear Hinge Definition
Source: Author, Using ETABS

Frame Hinge Property Data for S111 - P

Edit

Point	Force/SF	Disp/SF
E-	-1	-1000
D-	-1	-1000
C-	-1	-1000
B-	-1	0.
A	0.	0.
B	1.	0.
C	1.	1000.
D	1.	1000.
E	1.	1000.

☒ Hinge is Rigid Plastic
☒ Symmetric

Scaling for Force and Disp

☐ Use Yield Force Force SF Positive Negative 58.4

☒ Use Yield Disp Disp SF Positive Negative

Acceptance Criteria (Plastic Disp/SF)

Immediate Occupancy Positive Negative 100.

Life Safety 100.

Collapse Prevention 100.

Type

☒ Force - Displacement

☐ Stress - Strain Hinge Length

☐ Relative Length

OK

Cancel

Figure 162: Example Equivalent Strut Nonlinear Hinge Definition
Source: Author, Using ETABS

The dialog box titled "Assign Frame Hinges (Pushover)" contains a section labeled "Frame Hinge Data". It features two columns: "Hinge Property" and "Relative Distance". The "Hinge Property" column has a dropdown menu currently showing "18I55". Below this, there is a table with two rows of data. The first row has "18I55" in the "Hinge Property" column and "0.05" in the "Relative Distance" column. The second row has "18I55" in the "Hinge Property" column and "0.95" in the "Relative Distance" column. To the right of the table are three buttons: "Add", "Modify", and "Delete". At the bottom of the dialog are "OK" and "Cancel" buttons.

Hinge Property	Relative Distance
18I55	0.05
18I55	0.95

Figure 163: Sample Beam Nonlinear Hinges (Percentage of Distance)

Source: Author, Using ETABS

The dialog box titled "Assign Frame Hinges (Pushover)" contains a section labeled "Frame Hinge Data". It features two columns: "Hinge Property" and "Relative Distance". The "Hinge Property" column has a dropdown menu currently showing "C1". Below this, there is a table with two rows of data. The first row has "C1" in the "Hinge Property" column and "0.1" in the "Relative Distance" column. The second row has "C1" in the "Hinge Property" column and "0.9" in the "Relative Distance" column. To the right of the table are three buttons: "Add", "Modify", and "Delete". At the bottom of the dialog are "OK" and "Cancel" buttons.

Hinge Property	Relative Distance
C1	0.1
C1	0.9

Figure 164: Sample Column Nonlinear Hinges (Percentage of Distance)

Source: Author, Using ETABS

Figure 165: Sample Equivalent Strut Nonlinear Hinges (Percentage of Distance)
 Source: Author, Using ETABS

Modal Participating Mass Ratios

Edit View

Modal Participating Mass Ratios

	Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX
▶	1	2.106345	1.2796	78.3630	0.0000	1.2796	78.3630	0.0000	96.7586
	2	1.942977	76.7430	1.5876	0.0000	78.0226	79.9506	0.0000	1.9648
	3	1.207895	1.9421	0.7816	0.0000	79.9647	80.7322	0.0000	0.7138
	4	0.726588	3.5990	8.3761	0.0000	83.5637	89.1083	0.0000	0.1920
	5	0.701940	8.8525	3.4830	0.0000	92.4162	92.5913	0.0000	0.0857
	6	0.489715	0.0914	0.0051	0.0000	92.5076	92.5964	0.0000	0.0079
	7	0.436100	0.1268	3.6383	0.0000	92.6344	96.2347	0.0000	0.2444
	8	0.405380	3.7377	0.0497	0.0000	96.3720	96.2844	0.0000	0.0080
	9	0.298762	0.0021	2.4588	0.0000	96.3741	98.7432	0.0000	0.0066
	10	0.284470	1.3875	0.0105	0.0000	97.7617	98.7536	0.0000	0.0001
	11	0.272119	0.0755	0.0003	0.0000	97.8372	98.7539	0.0000	0.0004
	12	0.255227	0.0259	0.4413	0.0000	97.8631	99.1952	0.0000	0.0127

OK

Figure 166: Modal Analysis Output (k-in)

Source: Author, using ETABS

Figure 167: First Mode Shape, 3-D View
Source: Author, using ETABS

Figure 168: Second Mode Shape, 3-D View
Source: Author, using ETABS

Figure 169: Third Mode Shape, 3-D View
Source: Author, using ETABS

Figure 170: Fourth Mode Shape, 3-D View
Source: Author, using ETABS

Figure 171: Fifth Mode Shape, 3-D View
Source: Author, using ETABS

Figure 172: Sixth Mode Shape, 3-D View
Source: Author, using ETABS

Target Displacement Calculation (3.3.3.3.2), Example: St. Louis

C_0	1.5
-------	-----

(Table 3-2)

S_s	0.508
S_1	0.154

Soil Site Class: Class D (1.6.1.4)

F_a	1.4
F_v	2.2

(Table 1-4)

(Table 1-5)

S_{XS}	0.7112
S_{X1}	0.3388

(Eq. 1-4)

(Eq. 1-5)

β	0.05
B_1	1.00

(1.6.1.5.3)

(Eq. 1-13)

T_s	0.4764	seconds
T_0	0.0953	seconds

(Eq. 1-11)

(Eq. 1-12)

S_a	0.173958
-------	----------

(1.6.1.5.1)

K_i	361.6
K_e	345.5
T_i	1.9430

(3.3.3.2.6)

(Eq. 3-6)

T_e	1.9878	seconds
T_s	0.4764	seconds

(Eq. 3-13)

(Eq. 1-11)

C_1	1.0000
C_2	1.0000

S_a	0.173958	in/s ²
T_e^2	3.9512	
$4\pi^2$	39.48	
g	386.4	
δ_t	10.0911	

(Eq. 3-14)

Figure 173: Nonlinear Pushover Output, Dead Load
Source: Author, using ETABS

PUSHOVER CURVE

File

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0000	0.0000	2020	0	0	0	0	0	0	0	2020
1	0.0477	32626.8496	2020	0	0	0	0	0	0	0	2020

Figure 174: Nonlinear Pushover Data, Dead Load
Source: Author, using ETABS

Figure 175: Nonlinear Pushover Output, X-Direction
Source: Author, using ETABS

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0477	0.0000	1831	1	0	0	0	0	0	0	1832
1	0.7755	333.3255	1749	83	0	0	0	0	0	0	1832
2	2.3317	959.4955	1670	162	0	0	0	0	0	0	1832
3	3.7993	1173.9806	1645	187	0	0	0	0	0	0	1832
4	11.2966	1859.9801	1428	323	81	0	0	0	0	0	1832
5	20.7135	2524.5332	1305	233	260	26	8	0	0	0	1832
6	37.2188	3227.9968	1245	239	200	79	69	0	0	0	1832
7	57.1723	3896.4653	1233	244	123	111	121	0	0	0	1832
8	67.0854	4214.9185	1832	0	0	0	0	0	0	0	1832

Figure 176: Nonlinear Pushover Data, X-Direction
Source: Author, using ETABS

Force-Displacement Approximation: X-Direction

ETABS Output

Displ. (in)	Drift (%)	Force (k)		
0	0	0		
0.0477	0.00265	0		
0.7755	0.043083	333.3255	Ki	312.928
2.3317	0.129539	959.4955		
3.7993	0.211072	1173.981		
11.2966	0.627589	1859.98		
20.7135	1.15075	2524.533		
37.2188	2.067711	3227.997		
57.1723	3.176239	3896.465		
67.0854	3.726967	4214.919		

Approximation Using ASCE 41 3.3.3.2.5

Vi	0	Δ_i	0	K	0
Vy	1700	Δ_y	4	Ke	425
Vd	4215	Δ_d	67.1	a1Ke	39.85737
0.6Vy	1020	0.6 Δ_y	2.4	Ke	425

a1	0.093782
----	----------

Hand Approximation Calculations

E_m	330	ksi	f'_m	0.6	ksi
E_s	29000	ksi			

Stiffness Determination

Story	h_{Story} (in)	w_{Bay} (in)	238	Strut/Story	4		Lateral Fy
	θ	A (in ²)	L_{Strut} (in)	k (k/in)	Fy Strut		
Roof	151	0.565369	324	281.86	270.47	194.4	164.15
8th	120	0.467003	303	266.54	299.10	181.8	162.33
7th	120	0.467003	328	266.54	323.78	196.8	175.73
6th	195	0.686416	309	307.68	198.30	185.4	143.41
5th Mez	147	0.553294	309	279.74	263.86	185.4	157.74
5th	164	0.603356	324	289.03	250.83	194.4	160.08
4th Mez	128	0.49344	330	270.24	312.57	198	174.38
4th	150	0.562367	325	281.33	272.85	195	164.97
3rd	152	0.568359	428	282.40	355.25	256.8	216.43
2nd							
Mez	166	0.609032	426	290.17	325.92	255.6	209.64
2nd	120	0.467003	439	266.54	433.35	263.4	235.20
1st Mez	159	0.588972	427	286.23	340.39	256.2	213.03

Story	w_{Bay} (in)	214	Strut/Story	2		Lateral Fy
	θ	A (in ²)	L_{Strut} (in)	k (k/in)	Fy Strut	
Roof	0.614479	300	261.91	252.35	180	147.07
8th	0.511057	278	245.35	284.47	166.8	145.49
7th	0.511057	300	245.35	306.98	180	157.00
6th	0.738977	291	289.52	181.22	174.6	129.06
5th Mez	0.601891	286	259.62	246.98	171.6	141.44
5th	0.653886	301	269.61	232.10	180.6	143.35
4th Mez	0.539044	302	249.36	294.36	181.2	155.51
4th	0.611353	300	261.34	254.02	180	147.40
3rd	0.617592	396	262.49	330.91	237.6	193.71
2nd Mez	0.659748	397	270.84	302.00	238.2	188.21
2nd	0.511057	401	245.35	410.33	240.6	209.86
1st Mez	0.639	396	266.60	315.82	237.6	190.72

	w_{Bay} (in)	200	Strut/Story	2		
Story	θ	A (in ²)	L_{Strut} (in)	k (k/in)	Fy Strut	Lateral Fy
Roof	0.646693	287	250.60	240.72	172.2	137.43
8th	0.54042	263	233.24	273.61	157.8	135.31
7th	0.54042	284	233.24	295.46	170.4	146.12
6th	0.772741	280	279.33	169.58	168	120.29
5th Mez	0.633832	272	248.21	234.79	163.2	131.50
5th	0.686818	288	258.64	219.72	172.8	133.62
4th Mez	0.569313	287	237.45	282.96	172.2	145.04
4th	0.643501	287	250.00	242.46	172.2	137.76
3rd	0.64987	378	251.21	314.76	226.8	180.57
2nd Mez	0.692768	380	259.92	285.67	228	175.44
2nd	0.54042	380	233.24	395.33	228	195.51
1st Mez	0.671685	380	255.50	300.73	228	178.47

	w_{Bay} (in)	128	Strut/Story	1		
Story	θ	A (in ²)	L_{Strut} (in)	k (k/in)	Fy Strut	Lateral Fy
Roof	0.867649	226	197.95	157.53	135.6	87.68
8th	0.753151	195	175.45	195.20	117	85.36
7th	0.753151	211	175.45	211.22	126.6	92.36
6th	0.989928	236	233.26	100.54	141.6	77.70
5th Mez	0.854379	213	194.92	155.51	127.8	83.92
5th	0.908067	232	208.04	139.31	139.2	85.65
4th Mez	0.785398	217	181.02	197.80	130.2	92.07
4th	0.86437	225	197.19	158.66	135	87.63
3rd	0.870903	298	198.72	205.33	178.8	115.17
2nd Mez	0.913937	307	209.62	180.21	184.2	112.48
2nd	0.753151	283	175.45	283.29	169.8	123.88
1st Mez	0.892995	303	204.12	192.63	181.8	114.00

Story	h_{Story} (in)	I_{Column}	k (k/in)	Columns
Roof	151	50.25	5.079073	62
8th	120	50.25	10.11979	62
7th	120	108.74	21.89903	62
6th	195	76.32	3.581903	62
5th Mez	147	76.32	8.361142	62
5th	164	126.74	9.999111	60
4th Mez	128	126.74	21.03115	60
4th	150	126.74	13.0683	74
3rd	152	179.65	17.80228	80
2nd				
Mez	166	179.65	13.66728	80
2nd	120	179.94	36.23792	80
1st Mez	159	179.94	15.57814	80

$\Sigma k/\text{Story}$	F_y/Story	Load Distr.	Load	Load/Story	State	δ_{xe}
2225.53	1313.29	0.1527	230.70	230.70	Elastic	0.10
2507.77	1296.29	0.1397	211.04	441.75	Elastic	0.18
2711.21	1401.50	0.1294	195.42	637.17	Elastic	0.24
1595.33	1150.04	0.1190	179.80	816.97	Elastic	0.51
2174.50	1260.77	0.1022	154.41	971.38	Elastic	0.45
2046.25	1279.88	0.0896	135.27	1106.65	Elastic	0.54
2602.70	1390.68	0.0754	113.92	1220.57	Elastic	0.47
2243.02	1317.82	0.0644	97.26	1317.82	Yielded	0.59
2917.66	1729.44	0.0515	77.73	1395.55	Elastic	0.48
2659.23	1678.36	0.0384	57.94	1453.48	Elastic	0.55
3628.02	1875.39	0.0240	36.32	1489.81	Elastic	0.41
2787.28	1704.52	0.0137	20.70	1510.51	Elastic	0.54

Sum	17397.98	Sum	1510.51		SUM	5.05
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Figure 178: Nonlinear Pushover Output, Y-Direction
Source: Author, using ETABS

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E TOTAL
0	0.2612	0.0000	1803	1	0	0	0	0	0	1804
1	1.7367	604.7375	1763	41	0	0	0	0	0	1804
2	2.2672	790.2091	1723	81	0	0	0	0	0	1804
3	3.1703	917.4632	1651	153	0	0	0	0	0	1804
4	13.5618	1575.6578	1461	203	140	0	0	0	0	1804
5	33.4590	2237.8452	1358	209	88	91	58	0	0	1804
6	68.8088	2921.9927	1303	198	99	13	191	0	0	1804
7	119.7465	3581.9146	1274	191	75	61	203	0	0	1804
8	150.3376	3928.0576	1804	0	0	0	0	0	0	1804

Figure 179: Nonlinear Pushover Data, Y-Direction
Source: Author, using ETABS

Force-Displacement Approximation: Y-Direction

ETABS Output

Displ. (in)	Drift (%)	Force (k)		
0	0	0		
0.22612	0.012562	0	Ki	348.2107
1.7367	0.096483	604.7375		
2.2672	0.125956	790.2091		
3.1703	0.176128	917.4632		
13.5618	0.753433	1575.658		
33.459	1.858833	2237.845		
68.8088	3.822711	2921.993		
119.7465	6.652583	3581.915		
150.3376	8.352089	3928.058		

Approximation Using ASCE 41 3.3.3.2.5

Vi	0	Δ_i	0	K	0
Vy	1700	Δ_y	5	Ke	340
Vd	3930	Δ_d	150.3	a1Ke	15.34756
0.6Vy	1020	0.6 Δ_y	3	Ke	340

a1	0.04514
----	---------

Hand Analysis

E	330	ksi	f'_m	0.6	ksi
Stiffness Determination					

Story	h_{Story} (in)	w_{Bay} (in) θ	214 A (in ²)	Strut/Story L_{Strut} (in)	5 k (k/in)	Fy Strut	Lateral Fy
Roof	151	0.614479	300	261.91	252.35	180	147.07
8th	120	0.511057	278	245.35	284.47	166.8	145.49
7th	120	0.511057	300	245.35	306.98	180	157.00
6th	195	0.738977	291	289.52	181.22	174.6	129.06
5th Mez	147	0.601891	286	259.62	246.98	171.6	141.44
5th	164	0.653886	301	269.61	232.10	180.6	143.35
4th Mez	128	0.539044	302	249.36	294.36	181.2	155.51
4th	150	0.611353	300	261.34	254.02	180	147.40
3rd	152	0.617592	396	262.49	330.91	237.6	193.71
2nd Mez	166	0.659748	397	270.84	302.00	238.2	188.21
2nd	120	0.511057	401	245.35	410.33	240.6	209.86
1st Mez	159	0.639	396	266.60	315.82	237.6	190.72

w_{Bay} (in)	255	Strut/Story	2		
θ	A (in ²)	L_{Strut} (in)	k (k/in)	Fy Strut	Lateral Fy
0.534633	342	296.35	281.96	205.2	176.57
0.439843	322	281.82	308.68	193.2	174.81
0.439843	348	281.82	333.61	208.8	188.93
0.652847	323	321.01	209.52	193.8	153.95
0.522939	326	294.34	274.33	195.6	169.46
0.571536	340	303.18	261.79	204	171.58
0.465215	349	285.32	322.41	209.4	187.15
0.531724	342	295.85	283.41	205.2	176.87
0.537531	451	296.87	369.91	270.6	232.44
0.577064	448	304.27	341.26	268.8	225.27
0.439843	466	281.82	446.73	279.6	252.99
0.557541	449	300.51	355.03	269.4	228.60

Story	$\Sigma k/\text{Story}$	Yield Force/Story	Load Distr.	Applied Load	Total Load/Story	State	δ_{xe}
Roof	1825.67	1088.50	0.1527	190.95	190.95	Elastic	0.10
8th	2039.71	1077.06	0.1397	174.68	365.62	Elastic	0.18
7th	2202.12	1162.86	0.1294	161.75	527.37	Elastic	0.24
6th	1325.14	953.18	0.1190	148.81	676.18	Elastic	0.51
5th Mez	1783.59	1046.14	0.1022	127.80	803.98	Elastic	0.45
5th	1684.09	1059.89	0.0896	111.96	915.94	Elastic	0.54
4th Mez	2116.60	1151.82	0.0754	94.29	1010.23	Elastic	0.48
4th	1836.93	1090.72	0.0644	80.50	1090.73	Yielded	0.59
3rd	2394.36	1433.42	0.0515	64.33	1155.06	Elastic	0.48
2nd							
Mez	2192.55	1391.61	0.0384	47.95	1203.01	Elastic	0.55
2nd	2945.11	1555.26	0.0240	30.06	1233.08	Elastic	0.42
1st Mez	2289.17	1410.80	0.0137	17.13	1250.21	Elastic	0.55

Sum	14421.27	Sum	1250.21		SUM	5.10
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Figure 180: Hinged Structure, X-Axis Pushover, 3-D View
Source: Author, using ETABS

Figure 181: Hinged Structure, Y-Axis Pushover, 3-D View
Source: Author, using ETABS

Hand Calculated Life Safety Drift Limits for Lateral Strut Elements

**Line
1**

Story	Height (in)	w_{bay} (in)	238		Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
Roof	151	52.3	270.4662	0.19337	0.001281	0.25	0.001601	0.241712
8th	120	55.3	299.102	0.184887	0.001541	0.2	0.001849	0.221864
7th	120	55.3	323.7804	0.170795	0.001423	0.7	0.00242	0.290351
6th	195	47.9	198.2963	0.241558	0.001239	0.85	0.002292	0.446882
5th								
Mez	147	52.6	263.8608	0.199348	0.001356	0.8	0.002441	0.358826
5th	164	51	250.8252	0.203329	0.00124	0.8	0.002232	0.365992
4th								
Mez	128	54.5	312.5703	0.174361	0.001362	0.75	0.002384	0.305131
4th	150	52.4	272.8502	0.192047	0.00128	0.8	0.002305	0.345684
3rd	152	68.2	355.2482	0.191978	0.001263	0.5	0.001895	0.287968
2nd								
Mez	166	66.4	325.9187	0.203732	0.001227	0.5	0.001841	0.305598
2nd	120	72.3	433.3525	0.166839	0.00139	0.5	0.002085	0.250258
1st								
Mez	159	67.3	340.3855	0.197717	0.001244	0.8	0.002238	0.355891

w _{bay} (in)	200						
F _y (k)	k (k/in)	Δ _y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ _{LS} (in)	
45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381	
48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116	
48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483	
41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914	
45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515	
44.2	232.10	0.190434	0.001161	0.85	0.002148	0.352303	
47.8	294.36	0.162389	0.001269	0.75	0.00222	0.28418	
45.6	254.02	0.179513	0.001197	0.8	0.002154	0.323123	
59.3	330.91	0.179204	0.001179	0.5	0.001768	0.268805	
57.5	302.00	0.190394	0.001147	0.55	0.001778	0.295111	
63.4	410.33	0.154509	0.001288	0.7	0.002189	0.262666	
58.4	315.82	0.184914	0.001163	0.8	0.002093	0.332845	

w_{bay} (in)	128					
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
21.5	240.72	0.089317	0.000592	0.55	0.000917	0.138441
24.3	273.61	0.088813	0.00074	0.6	0.001184	0.1421
24.3	295.46	0.082246	0.000685	0.9	0.001302	0.156267
18.3	169.58	0.107912	0.000553	1.05	0.001134	0.22122
21.9	234.79	0.093275	0.000635	0.95	0.001237	0.181887
20.5	219.72	0.093301	0.000569	1	0.001138	0.186603
23.5	282.96	0.083051	0.000649	0.9	0.001233	0.157797
21.6	242.46	0.089088	0.000594	1	0.001188	0.178175
28	314.76	0.088956	0.000585	1	0.00117	0.177913
26.6	285.67	0.093115	0.000561	0.75	0.000982	0.162951
31.7	395.33	0.080186	0.000668	0.9	0.00127	0.152354
27.3	300.73	0.090779	0.000571	1	0.001142	0.181558

w_{bay} (in)	214					
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
41.5	157.53	0.263442	0.001745	0.25	0.002181	0.329303
44.6	195.20	0.228483	0.001904	0.25	0.00238	0.285604
44.6	211.22	0.211157	0.00176	0.8	0.003167	0.380083
44.6	100.54	0.443604	0.002275	0.9	0.004322	0.842847
37.2	155.51	0.239213	0.001627	0.85	0.003011	0.442544
41.9	139.31	0.300762	0.001834	0.85	0.003393	0.55641
40.2	197.80	0.203239	0.001588	0.8	0.002858	0.36583
41.6	158.66	0.2622	0.001748	0.85	0.003234	0.485071
54.1	205.33	0.263478	0.001733	0.85	0.003207	0.487435
52.3	180.21	0.290215	0.001748	0.55	0.00271	0.449833
58.3	283.29	0.205795	0.001715	0.8	0.003087	0.370432
53.2	192.63	0.27618	0.001737	0.85	0.003213	0.510933

**Line
3**

		W_{bay} (in)	128					
Story	Height (in)	F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
Roof	151	21.5	270.4662	0.079492	0.000526	1	0.001053	0.158985
8th	120	24.3	299.102	0.081243	0.000677	0.9	0.001286	0.154362
7th	120	24.3	323.7804	0.075051	0.000625	0.9	0.001188	0.142597
6th	195	18.3	198.2963	0.092286	0.000473	0.9	0.000899	0.175344
5th								
Mez	147	21.9	263.8608	0.082998	0.000565	1	0.001129	0.165997

W_{bay} (in)	200						
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)	
41.5	252.35	0.164453	0.001089	0.25	0.001361	0.205567	
44.6	284.47	0.156783	0.001307	0.25	0.001633	0.195979	
44.6	306.98	0.145286	0.001211	0.8	0.002179	0.261515	
37.2	181.22	0.205276	0.001053	1	0.002105	0.410552	
41.9	246.98	0.169646	0.001154	0.85	0.002135	0.313846	

**Line
8**

		W_{bay} (in)	238					
Story	Height (in)	F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
Roof	151	52.3	270.4662	0.19337	0.001281	0.25	0.001601	0.241712
8th	120	55.3	299.102	0.184887	0.001541	0.2	0.001849	0.221864
7th	120	55.3	323.7804	0.170795	0.001423	0.7	0.00242	0.290351
6th	195	47.9	198.2963	0.241558	0.001239	0.85	0.002292	0.446882
5th								
Mez	147	52.6	263.8608	0.199348	0.001356	0.8	0.002441	0.358826
5th	164	51	250.8252	0.203329	0.00124	0.8	0.002232	0.365992
4th								
Mez	128	54.5	312.5703	0.174361	0.001362	0.75	0.002384	0.305131
4th	150	52.4	272.8502	0.192047	0.00128	0.8	0.002305	0.345684
3rd	152	68.2	355.2482	0.191978	0.001263	0.5	0.001895	0.287968
2nd								
Mez	166	66.4	325.9187	0.203732	0.001227	0.5	0.001841	0.305598
2nd	120	72.3	433.3525	0.166839	0.00139	0.7	0.002364	0.283626
1st								
Mez	159	67.3	340.3855	0.197717	0.001244	0.8	0.002238	0.355891

w_{bay} (in)	200					
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381
48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116
48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483
41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914
45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515
44.2	232.10	0.190434	0.001161	0.85	0.002148	0.352303
47.8	294.36	0.162389	0.001269	0.8	0.002284	0.292299
45.6	254.02	0.179513	0.001197	0.8	0.002154	0.323123
59.3	330.91	0.179204	0.001179	0.8	0.002122	0.322567
57.5	302.00	0.190394	0.001147	0.85	0.002122	0.352229
63.4	410.33	0.154509	0.001288	0.75	0.002253	0.270391
58.4	315.82	0.184914	0.001163	0.85	0.002152	0.342091

w_{bay} (in)	128					
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
21.5	240.72	0.089317	0.000592	0.7	0.001006	0.151838
24.3	273.61	0.088813	0.00074	0.6	0.001184	0.1421
24.3	295.46	0.082246	0.000685	0.9	0.001302	0.156267
18.3	169.58	0.107912	0.000553	1.05	0.001134	0.22122
21.9	234.79	0.093275	0.000635	1	0.001269	0.186551
20.5	219.72	0.093301	0.000569	1	0.001138	0.186603
23.5	282.96	0.083051	0.000649	0.9	0.001233	0.157797
21.6	242.46	0.089088	0.000594	1	0.001188	0.178175
28	314.76	0.088956	0.000585	1	0.00117	0.177913
26.6	285.67	0.093115	0.000561	1	0.001122	0.18623
31.7	395.33	0.080186	0.000668	0.9	0.00127	0.152354
27.3	300.73	0.090779	0.000571	0.8	0.001028	0.163402

w_{bay} (in)	214					
F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
41.5	157.53	0.263442	0.001745	0.25	0.002181	0.329303
44.6	195.20	0.228483	0.001904	0.25	0.00238	0.285604
44.6	211.22	0.211157	0.00176	0.8	0.003167	0.380083
44.6	100.54	0.443604	0.002275	0.9	0.004322	0.842847
37.2	155.51	0.239213	0.001627	0.85	0.003011	0.442544
41.9	139.31	0.300762	0.001834	0.85	0.003393	0.55641
40.2	197.80	0.203239	0.001588	0.8	0.002858	0.36583
41.6	158.66	0.2622	0.001748	0.85	0.003234	0.485071
54.1	205.33	0.263478	0.001733	0.55	0.002687	0.408391
52.3	180.21	0.290215	0.001748	0.55	0.00271	0.449833
58.3	283.29	0.205795	0.001715	0.8	0.003087	0.370432
53.2	192.63	0.27618	0.001737	0.85	0.003213	0.510933

Hand Calculated Life Safety Drift Limits for Lateral Strut Elements, Y-Direction

Line A

Story	Height (in)	w_{bay} (in)	255			LS Drift (%)	LS Drift	Δ_{LS} (in)
		F_y (k)	k (k/in)	Δ_y (in)	Yield Drift			
Roof	151	57	281.9589	0.202157	0.001339	0.25	0.001673	0.252696
8th	120	60	308.6842	0.194373	0.00162	0.2	0.001944	0.233248
7th	120	60	333.609	0.179851	0.001499	0.7	0.002548	0.305747
6th	195	52.7	209.5197	0.251528	0.00129	0.85	0.002386	0.465326
5th								
Mez	147	57.4	274.3338	0.209234	0.001423	0.75	0.002491	0.36616
5th	164	55.8	261.7888	0.213149	0.0013	0.8	0.002339	0.383668
4th								
Mez	128	59.8	322.4119	0.185477	0.001449	0.7	0.002463	0.315311
4th	150	57.1	283.4146	0.201472	0.001343	0.75	0.002351	0.352575
3rd	152	74.5	369.907	0.201402	0.001325	0.5	0.001988	0.302103
2nd								
Mez	166	72.7	341.2632	0.213032	0.001283	0.5	0.001925	0.319548
2nd	120	78.4	446.7292	0.175498	0.001462	0.4	0.002047	0.245697
1st								
Mez	159	73.6	355.0306	0.207306	0.001304	0.5	0.001956	0.310959

Story	w_{bay} (in)	214		Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
	F_y (k)	k (k/in)	Δ_y (in)				
Roof	45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381
8th	48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116
7th	48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483
6th	41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914
5th Mez	45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515
5th	44.2	232.10	0.190434	0.001161	0.85	0.002148	0.352303
4th Mez	47.8	294.36	0.162389	0.001269	0.8	0.002284	0.292299
4th	45.6	254.02	0.179513	0.001197	0.8	0.002154	0.323123
3rd	59.3	330.91	0.179204	0.001179	0.55	0.001827	0.277766
2nd							
Mez	57.5	302.00	0.190394	0.001147	0.55	0.001778	0.295111
2nd	63.5	410.33	0.154753	0.00129	0.75	0.002257	0.270818
1st Mez	58.4	315.82	0.184914	0.001163	0.85	0.002152	0.342091

**Line
D**

		w_{bay} (in)	255					
Story	Height (in)	F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
Roof	151	57	281.9589	0.202157	0.001339	0.25	0.001673	0.252696
8th	120	60	308.6842	0.194373	0.00162	0.2	0.001944	0.233248
7th	120	60	333.609	0.179851	0.001499	0.7	0.002548	0.305747
6th	195	52.7	209.5197	0.251528	0.00129	0.85	0.002386	0.465326
5th Mez	147	57.4	274.3338	0.209234	0.001423	0.75	0.002491	0.36616

	w _{bay} (in)	214					
Story	F _y (k)	k (k/in)	Δ _y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ _{LS} (in)
Roof	45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381
8th	48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116
7th	48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483
6th	41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914
5th Mez	45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515

**Line
G**

		w_{bay} (in)	255					
Story	Height (in)	F_y (k)	k (k/in)	Δ_y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
Roof	151	57	281.9589	0.202157	0.001339	0.25	0.001673	0.252696
8th	120	60	308.6842	0.194373	0.00162	0.2	0.001944	0.233248
7th	120	60	333.609	0.179851	0.001499	0.7	0.002548	0.305747
6th	195	52.7	209.5197	0.251528	0.00129	0.85	0.002386	0.465326
5th Mez	147	57.4	274.3338	0.209234	0.001423	0.75	0.002491	0.36616

	w _{bay} (in)	214					
Story	F _y (k)	k (k/in)	Δ _y (in)	Yield Drift	LS Drift (%)	LS Drift	Δ _{LS} (in)
Roof	45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381
8th	48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116
7th	48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483
6th	41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914
5th Mez	45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515

Line
J

Story	Height (in)	w_{bay} (in)	255			LS Drift (%)		
		F_y (k)	k (k/in)	Δ_y (in)	Yield Drift		LS Drift	Δ_{LS} (in)
Roof	151	57	281.9589	0.202157	0.001339	0.25	0.001673	0.252696
8th	120	60	308.6842	0.194373	0.00162	0.2	0.001944	0.233248
7th	120	60	333.609	0.179851	0.001499	0.7	0.002548	0.305747
6th	195	52.7	209.5197	0.251528	0.00129	0.85	0.002386	0.465326
5th								
Mez	147	57.4	274.3338	0.209234	0.001423	0.85	0.002633	0.387083
5th	164	55.8	261.7888	0.213149	0.0013	0.8	0.002339	0.383668
4th								
Mez	128	59.8	322.4119	0.185477	0.001449	0.7	0.002463	0.315311
4th	150	57.1	283.4146	0.201472	0.001343	0.85	0.002485	0.372722
3rd	152	74.5	369.907	0.201402	0.001325	0.5	0.001988	0.302103
2nd								
Mez	166	72.7	341.2632	0.213032	0.001283	0.5	0.001925	0.319548
2nd	120	78.4	446.7292	0.175498	0.001462	0.4	0.002047	0.245697
1st								
Mez	159	73.6	355.0306	0.207306	0.001304	0.5	0.001956	0.310959

Story	w_{bay} (in)	214		Yield Drift	LS Drift (%)	LS Drift	Δ_{LS} (in)
	F_y (k)	k (k/in)	Δ_y (in)				
Roof	45.5	252.35	0.180304	0.001194	0.25	0.001493	0.225381
8th	48.5	284.47	0.170493	0.001421	0.25	0.001776	0.213116
7th	48.5	306.98	0.15799	0.001317	0.75	0.002304	0.276483
6th	41.1	181.22	0.226797	0.001163	0.9	0.00221	0.430914
5th Mez	45.9	246.98	0.185842	0.001264	0.8	0.002276	0.334515
5th	44.2	232.10	0.190434	0.001161	0.8	0.00209	0.342781
4th Mez	47.8	294.36	0.162389	0.001269	0.8	0.002284	0.292299
4th	45.6	254.02	0.179513	0.001197	0.5	0.001795	0.269269
3rd	59.3	330.91	0.179204	0.001179	0.8	0.002122	0.322567
2nd							
Mez	57.5	302.00	0.190394	0.001147	0.55	0.001778	0.295111
2nd	63.5	410.33	0.154753	0.00129	0.75	0.002257	0.270818
1st Mez	58.4	315.82	0.184914	0.001163	0.85	0.002152	0.342091