DESIGN COMPARISON OF HYBRID MASONRY TYPES FOR SEISMIC LATERAL FORCE RESISTANCE FOR LOW-RISE BUILDINGS

by

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Abstract

The term *hybrid masonry* describes three variations of a lateral force resisting system that utilizes masonry panels inside steel framing to resist lateral loads from wind or earthquakes. The system originates from the rich history of masonry in the construction industry and is currently used in low-rise, low-seismic, wind-governed locations within the United States. Considerable research is focused on hybrid systems to prove their validity in high-seismic applications. The three variations of hybrid masonry are known by number.

Type I hybrid masonry utilizes the masonry panel as a non-load-bearing masonry shear wall. Shear loads from the diaphragm are transferred into the beam, through metal plates, and over an air gap to the top of the masonry panel. The masonry panel transfers the shear to the beam below the panel using compression at the toe of the wall and tension through the reinforcement that is welded to the beam supporting the masonry. Steel framing in this system is designed to resist all gravity loads and effects from the shear wall.

Type II hybrid masonry utilizes the masonry as a load-bearing masonry shear wall. The masonry wall, which is constructed from the ground up, supports the floor live loads and dead load of the wall, as well as the lateral seismic load. Shear is transferred from the diaphragm to the steel beam and into the attached masonry panel via shear studs. The masonry panel transfers the seismic load using compression at the toe and opposite corner of the panel.

Type III hybrid masonry also utilizes the masonry panel as a load-bearing masonry shear wall, but the load transfer mechanisms are more complicated since the panel is attached to the surrounding steel framing on all four sides of the panel.

This study created standard building designs for hybrid systems and a standard moment frame system with masonry infill in order to evaluate the validity of Type I and II hybrid masonry. The hybrid systems were compared to the standard of a moment frame system based on constructability, design, and economics.

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Chapter 1 - Hybrid Masonry Background

The construction industry is continuously evolving. Means and methods, materials, and design are constantly being refined to meet current demand. Over time, primary materials such as timber, masonry, steel, and concrete have been incorporated into the building industry. However, the push to build taller, thinner, more irregular buildings has exposed each material's strengths and weaknesses. Naturally, designers worked to improve the qualities of individual materials, but they also found that these primary materials could be used together in order to overcome each material's unique weaknesses. This collaborative use of materials or systems is the foundation of the concept of hybrid masonry. Hybrid masonry utilizes masonry shear walls within steel framing for lateral resistance. The hybrid lateral system is discussed in detail throughout the report, but the following sections describe the history of the innovation.

Historical Masonry and Steel Relationships

Masonry is one of the oldest, most traditional types of building materials. Initially, masonry walls were unreinforced gravity bearing walls or columns that acted primarily in compression to resist overhead weight. This system worked well when buildings were relatively short. However, as cities grew rapidly throughout the nineteenth and twentieth centuries, increasingly taller buildings were required in order to efficiently utilize limited land (Walkowicz, 2010). Consequently, increasingly taller buildings required thicker masonry walls, but thick walls consumed valuable space. Therefore, steel began to be used with the masonry to help reclaim some of the lost space.. Several iterations of masonry and steel walls led to the current innovation of hybrid masonry.

Masonry-Bearing Wall Systems

Unreinforced masonry bearing walls are the most basic form of masonry, and the principles of masonry-bearing walls are the same whether the material is stone, adobe, or modern bricks and blocks. Masonry is strong in compression but very weak in tension. Interior and exterior bearing walls were designed to use compression to resist loads from gravity while successfully preventing the elements from entering the building. Height to thickness ratios were governed by code. Although the masonry-bearing walls were not specifically designed for lateral resistance, buildings built to these ratios had sufficient wall length and thickness to resist the required shear load. The masonry-bearing wall system performed very well in low- to mid-rise buildings. However, in tall buildings, the weight of the building and induced tension by lateral loads required very thick walls. For example, the 16-story Monadock building in Chicago, Illinois has exterior bearing walls measuring up to 6 ft in thickness. The Mondadock building also contains the first masonry adaptation, the replacement of interior bearing walls with wrought iron columns, thereby saving significant space and prompting development of the masonry-steel system, caged and skeletal frame systems.

Caged and Skeletal Frame Systems

Exterior bearing walls risked total replacement once iron successfully supplanted interior bearing walls. However, elements such as wind and water needed to be prevented from entering the building, so wrought iron and later steel-frame buildings with exoskeletons of masonry were developed. This steel system enclosed in masonry is known as a caged or skeletal frame system. Unfortunately, though, the masonry only carried its self-weight and no floor weights. The masonry often participated in the lateral system as well because braced and moment frames were not well developed or common at this time, but the masonry walls tied to the exterior unintentionally acted as shear walls to supplement the lateral capacity for caged and skeletal frame buildings (Walkowicz, 2010). Although this system allowed a significant decrease in wall width, it limited the size and quantity of openings in exterior walls. The desire for additional openings led to the masonry-steel system, transitional wall building systems.

Transitional Wall Building Systems

Transitional walls were similar to caged walls, but in transitional walls a steel frame supported the masonry at each level . The masonry was laid tight to the beams and columns of the steel frame, and the main masonry rigidly supported the decorative interior and exterior veneers. Masonry walls for transitional wall systems unintentionally resisted gravity loads and lateral loads. However, without any considerations for movement between the frame, masonry, and veneer, considerable serviceability issues often occurred. Many exterior and interior veneers experienced significant cracking, bulging, and spalling due to differential movement between the masonry wythes. The interface between the masonry and steel also experienced concentrated cracking and crushing. In addition, moisture negatively impacted this system because the porous masonry allowed water to penetrate into the wall. Transitional walls could be a single wythe thick because they were supported at every level, Wall thinness and subsequent cracking allowed water to penetrate to the steel, causing corrosion. Therefore, the cavity wall system was devised to mitigate the cracking and corrosion issues of the transitional wall system.

Cavity Wall System

The cavity wall system is the primary type of masonry construction at this time. In addition to architectural masonry sections, the cavity wall system consists of a frame building that resists all gravity and lateral loads. Typical cavity wall construction is shown in Figure 1. The masonry has a main layer, generally comprised of concrete masonry units (CMUs), that is isolated from the steel frames due to strategic gaps and flexible joints. An air gap and insulation separate the exterior veneer from the main wythe. The separation and barriers prevent corrosion-causing water from reaching the steel. Although the isolation of this system prevents cracking, it also reduces the functionality of the masonry. In the cavity wall system, no structural benefits are gained by the substantial weight of the masonry. The proposed improvement to this problem is hybrid masonry.

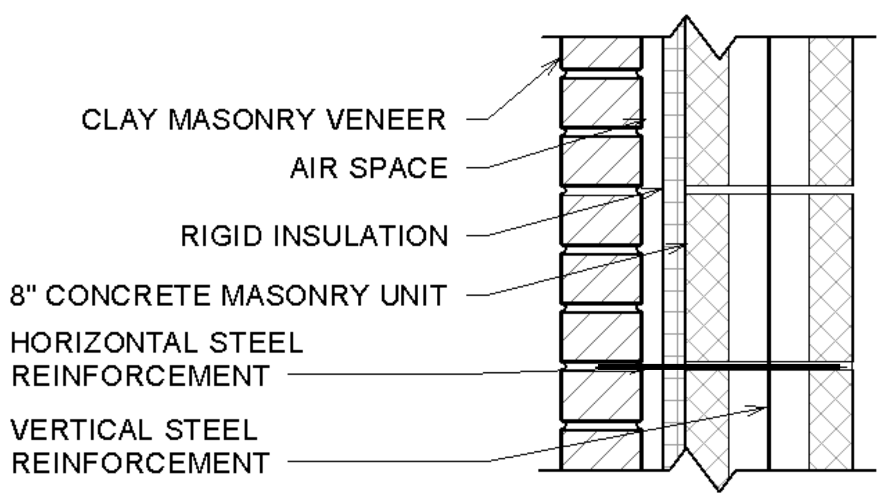


Figure 1 - Cavity Wall Section

Innovative Masonry and Steel Relationships

Each previous wall system had advantages and disadvantages, and each system innovatively utilized masonry to overcome previous obstacles. The next innovation in masonry is hybrid masonry. Hybrid masonry utilizes masonry shear walls and a steel frame structure for use as a lateral system in lieu of typical braced frames or moment frames. A reinforced masonry panel, typically CMU, is laid in a steel bay in plane with the columns and beams. Depending on the type of hybrid masonry (discussed in following sections), various connections and gaps are used between the steel and masonry to control the transfer of forces between the masonry and steel. The methods and locations of connections determine the type of loads and method of transfer to the masonry panel. Lateral loads are always transferred to the panel, while gravity loads may or may not be transferred to the panel depending on the type of hybrid wall. Gaps prevent unwanted load transfer and confinement that could cause undesirable cracking and crushing of the masonry. The masonry panel transfers loads into the beam below it through compression and/or tension at the base of the panel, depending on the hybrid type and base connections. Further description of hybrid masonry is included in Chapter 2.

Similar to any structural system, hybrid masonry is better suited for certain situations. Maximum system benefits are seen for projects that already use masonry walls. Structures such as schools, healthcare facilities, warehouses, retail developments, and offices are often constructed utilizing masonry for its familiarity, durability, and ease of construction which makes them good candidates for hybrid masonry use. Hybrid masonry is currently used in low- to mid- rise buildings in low and moderate seismic zones. A high-rise building is defined as a structure with at least one occupy able floor 75 ft above fire truck access (IBC, 2012); mid and low-rise buildings are less than high-rise. Hybrid masonry was used to build the Sis and Herman Dupre Science Complex at Saint Vincent College in Latrobe, Pennsylvania and Garden Hills Elementary School in Champaign, Illinois. These buildings are seismic design category B and C, respectively. Both design teams for the projects recognized the advantages of hybrid masonry and successfully applied the system to their projects. Hybrid masonry is proposed to be simple to construct while being an economically advantageous, well-performing structural solution.

Chapter 2 - Types of Hybrid Masonry

Hybrid masonry is classified into three main types according to loads transferred to the masonry panel and methods used to transfer those loads. Table 1 distinguishes the types of hybrid masonry. Type I hybrid masonry resists only lateral loads; the lateral loads are transferred to the top of the masonry panel through shear from the beam. Type II hybrid masonry resists lateral and gravity loads through shear transfer as well as tension and compression between the beam and top of the masonry panel. Type III hybrid masonry also resists lateral and gravity loads through shear, compression, and tension at the interfaces of the masonry panel with the beam and columns. The mechanics and construction of Type I, II, and III hybrid masonry systems is discussed in greater detail in the following sections.

Table 1 - Hybrid Masonry Summary

	Loads Resisted		Gaps	
Hybrid Masonry	Lateral	Gravity	Top of Panel	Side of Panel
Type I	yes	no	yes	yes
Type II	yes	yes	no	yes
Type III	yes	yes	no	no

Type I

Type I hybrid masonry is the first of the hybrid masonry family. This system utilizes a steel frame, reinforced masonry panel and steel plate connectors to resist lateral loads. In this system, the reinforced masonry panel only accepts lateral loads in the form of shear through plate connectors between the beam and panel. An elevation view of the system is shown in Figure 2. Callout 1 points to a steel beam in the system; the steel beam collects shear from the diaphragm and supports the floor, framing, and masonry wall overhead. Connectors in callout 2 are bolted or welded to the beam. The connector is the defining component of Type I hybrid masonry, and the primary function of the plate connector is to transfer lateral shear across the gap between the beam and the reinforced masonry panel. The connector is comprised of two identical plates, the shape of which may vary, but the plates always includes a slotted hole for through bolting to the reinforced masonry panel. The slot prevents the connector from transferring gravity loads to the reinforced masonry. The reinforced masonry wall is shown by callout 3. In Type I, the CMU wall supports veneer, protects occupants from the elements, and resists shear. In multilevel applications the reinforced masonry walls also transfer shear to the beam at the base of the individual panel. Callout 4 refers to the steel columns of the system. A free-body diagram for the masonry panel within the hybrid masonry frame is shown in Figure 3, and loads transferred to an example beam are shown in Figure 4.

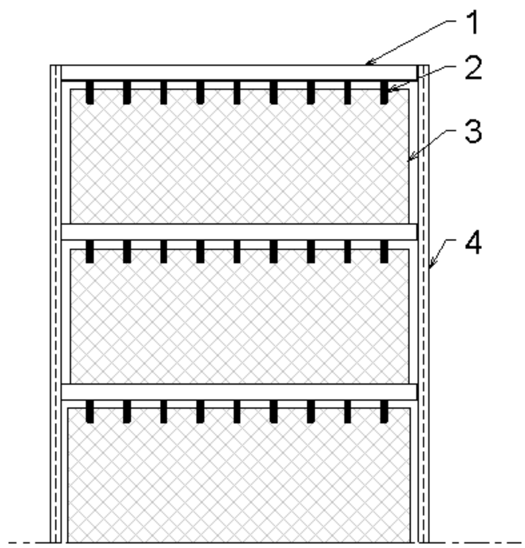


Figure 2 - Type I Hybrid Masonry Elevation

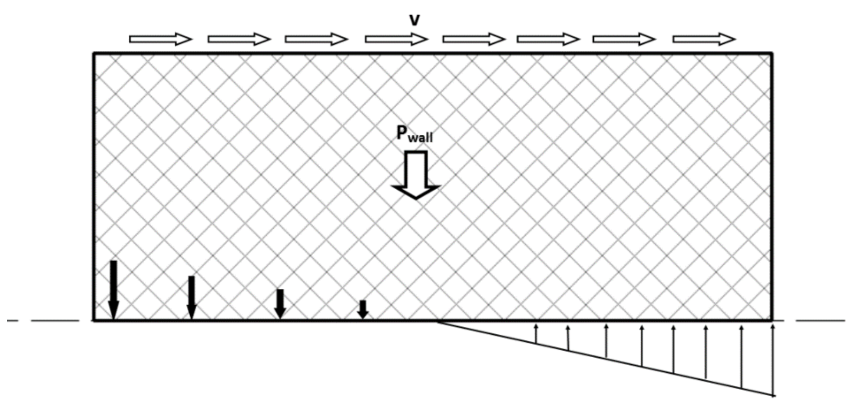


Figure 3 - Type I Hybrid Masonry Panel Free-Body Diagram

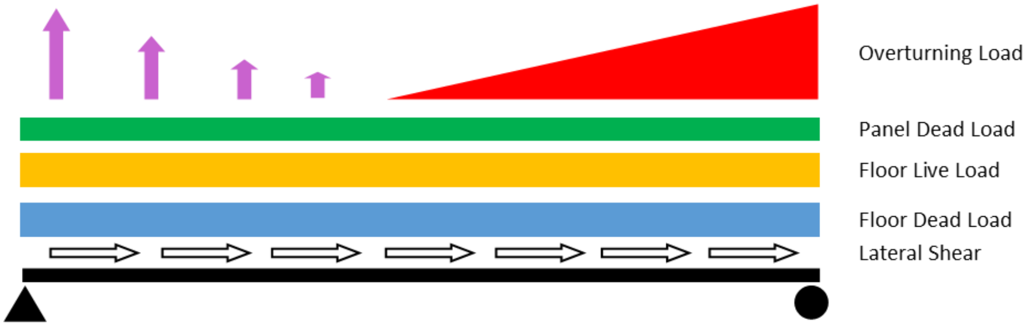


Figure 4 - Type I Hybrid Masonry Beam Loading

When the masonry panel is loaded at the top from the connectors it creates an overturning moment in the panel, as shown by the panel load diagram in Figure 4. Compression and tension forces in the reinforcement at the base of the panel distribute moment to the beam from the panel. The composite concept is similar to that of a moment within a concrete beam: the masonry is under compression on one side of the panel (i.e., the right side of the panel), and the opposite side of the panel is under tension. The compression load may be idealized as a triangular load, as shown on the right of the Figures 3 and 4. Because masonry is weak in tension, the steel reinforcement within the panel resists the tension force. The tension forces are represented by vertical arrows on the left sides of Figures 3 and 4. Tension, compression, floor loads, and the weight of the masonry panel are transferred as vertical shear through the beam to the columns through simple pin connections. However, gravity loads from the floor cannot be transferred to the masonry panel in Type I hybrid masonry. Finally, axial compression or tension transfers the loads to the foundation and into the earth. Determination of the exact quantity of these forces is discussed in Chapter 3, and a design example is shown in Appendix B. As demonstrated in Figure 4 in blue and yellow, the beam experiences dead and live loads, respectively, from the corresponding floor. The beam also experiences compression resulting from the panel overturning on one side of the beam, as shown by the red triangle. The purple upward arrows represent tension from the reinforcement from the overturning of the masonry panel. Supports of the diagram are shown as a triangle and circle to represent the simple span condition of the beam.

System ductility is critical in order for a building system to survive a seismic event. Ductility is the ability of the system to dissipate energy through plastic or permanent deformations (SEOAC Seismology Committee, 2008). Plastic deformation must be planned or designed in a controlled area to prevent global failure or progressive collapse of a structure. The location of plastic deformation in Type I hybrid masonry depends on the system's key component, the connector plate. The design and strength of the connector plate determines if the deformation will be localized in the fuse connector or the reinforced masonry panel. Two examples of connectors are illustrated in Figure 5. Connector B is a fuse-type connector designed for a capacity of shear less than the strength of the masonry wall. When that shear level is reached, the thinner portion of the steel plate deforms significantly. However, deformations are concentrated in this reduced area, similar to designing a reduced beam section in a steel moment frame. One advantage of the fuse connector is that deformed links may be efficiently removed and replaced following a seismic event. Connector A is a link-type connector designed to be stronger than the reinforced masonry panel which includes masonry, mortar, and reinforcement, resulting in concentrated deformation in the masonry panel. The panel dissipates energy through masonry cracking, masonry crushing at the base of the panel, and reinforcement yielding within the panel. After a seismic event, the masonry panel could be removed and replaced for continued use of the structure. Since localizing deformations in the masonry panel is possible in all three types of hybrid masonry, the link-type connector, or Connector A in Figure 5, was used in this study to reduce the differences between each case in the experiment. Additional information about connections for Type I hybrid masonry is provided in University of Hawaii reports (Ozaki-Train, Johnson, & Robertson, 2011).

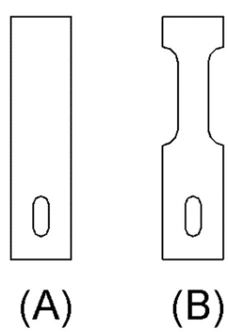


Figure 5 - Shear Plate Connectors

Construction of a Type I hybrid masonry panel is typically straightforward with a couple of nuances. Steel beams and columns are erected according to current design practices, and only simple pin connections are required for steel connections, preventing costly field welding. Bases of the frames may also be pin connections, making the baseplates, anchor rods, and foundations less complicated and smaller than moment-resisting bases. The steel does not have any resistance to lateral loads until the masonry and links are constructed. Basic construction of the reinforced masonry panels is similar to typical masonry infill construction, with the exception that all reinforcement must be welded to the beam supporting the masonry panel to transmit tension from the reinforcement into the beam. At the top of the panel, the connectors may be bolted or welded to the beam, and the other end of the connector is through-bolted to the masonry.

Type II

Type II hybrid masonry utilizes a steel frame, reinforced masonry panels, and steel stud shear connectors to resist lateral loads. The reinforced masonry panel of this system accepts lateral loads from the steel frame and may participate in load sharing of the gravity loads. An elevation and section of the system is shown in Figure 6. Callout 1 points to a steel beam in the system; the beam collects shear from the diaphragm and supports the floor, framing, and masonry wall overhead. Shear studs are referenced by callout 2; the studs are welded to the beam and embedded in grout to connect to the reinforced masonry panel. The reinforced masonry panel, identified by callout 3, is constructed of CMU, steel reinforcing, and grout. It can support veneer, protect occupants from the elements, and resist shear and gravity loads. In multilevel applications the reinforced masonry panel also transfers shear to beams above and below the individual panel.

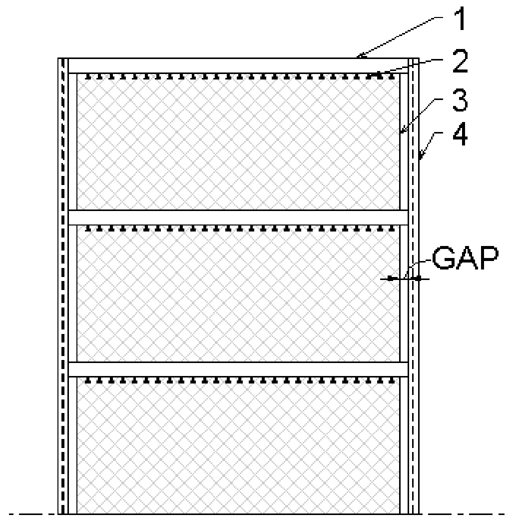


Figure 6 - Type II Hybrid Masonry Elevation

Masonry panels distribute loads to the surrounding beams in one of two ways depending on the connection of the reinforcement to the beam. The first transfer method is nearly identical to the way Type I hybrid masonry transfers loads to the beam using tension and compression at the base of the panel, requiring each piece of reinforcement to be welded to the beam supporting the masonry panel. The only difference is that, for Type II hybrid masonry, the gravity load may be shared by the masonry. The second transfer method does not require reinforcement in the masonry panel to be welded to the beam supporting the masonry panel. All loads are transferred by compression to the beam above and below the masonry panel, as shown in Figure 7.

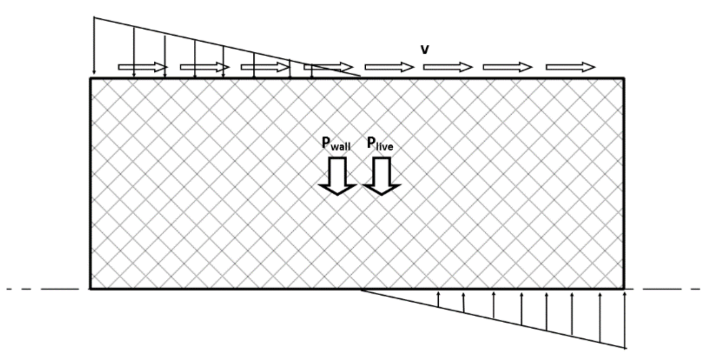


Figure 7 - Type II Hybrid Masonry Panel Free-Body Diagram

Shear load is transferred to the top of the masonry panel from the beam through the shear studs, creating an overturning moment in the panel. The overturning moment is resisted by the weight of the wall, the gravity loads the wall is designed to share with the steel frame, and compression against the beams surrounding the panel. The masonry panel is in compression at one end of the base of the panel, as indicated in the lower right corner in Figure 7. The opposite corner also transfers compression to the beam above the individual panel, as shown in the upper left corner of Figure 7. If shear loading was reversed, the opposite corners would be in compression. Resulting loads on a typical beam are shown in Figure 8.

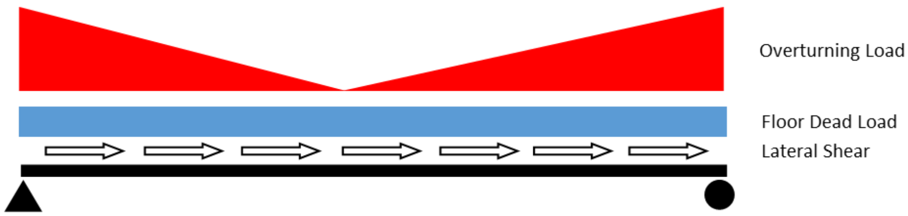


Figure 8 - Type II Hybrid Typical Beam Load Diagram

The load diagram in Figure 8 is similar to the previously discussed hybrid masonry beam loadings. The beam experiences dead loads from the corresponding floor, as shown in blue in the figure, and the beam experiences compression on each side of the beam, as indicated by the red triangles. Live loads are not shown in Figure 8 because the system may be designed so that the full-height masonry panels carry their self-weight and the live load from the floor.

As mentioned for Type I, system ductility, which is critical in order for a system to survive a seismic event, is created when a system permanently deforms in order to absorb the seismic energy. For Type II systems, the masonry panel dissipates energy through masonry cracking, masonry crushing at the base of the panel, and yielding of the reinforcement within the panel. After a seismic event, the masonry panel may be removed and replaced to allow continued use of the structure.

The construction of Type II hybrid masonry is relatively straight forward. The beams and columns of Type II hybrid masonry require only simple shear connections and pinned bases, thereby providing the same benefits as discussed for Type I. Although reinforcement welding at the base of the masonry panel may or may not be necessary depending on the desired load transfer method, the shear studs do require welding to the underside of the beam. Welding of shear studs is a common practice due to the use of composite beams. Because these studs must be embedded into the wall, International Masonry Institute (IMI) suggests laying the block for the wall within one course of the underside of the beam, affixing plywood to each side of the wall to act as a form, and then pumping grout into the space (IMI Technology Brief 02.13.02, 2010). After the grout has set, the plywood may be removed to reveal a flat surface with the studs embedded inside. This method also ensures that the beam bears directly on the top of the wall to allow load sharing of the gravity loads. Sequencing of the project is critical in order to take advantage of the load-sharing capabilities of the masonry panels. The masonry panel below each floor must be completed and have reached design strength before any loads can be introduced to the floors above the panel in question.

Type III

Type III hybrid masonry utilizes a steel frame, reinforced masonry panel, and steel stud shear connectors to resist lateral loads. The reinforced masonry panel of this system accepts lateral loads from the steel frame and may also participate in load sharing of the gravity loads. An elevation of the system is shown in Figure 9. Callout 1 points to a steel beam in the system; the steel beam collects shear from the diaphragm and supports the floor, framing, and masonry wall on the associated level. Callout 2 references shear studs; the studs are welded to the beam and columns surrounding the reinforced masonry panel shown by callout 3. The reinforced masonry panel, which is constructed of CMU, steel reinforcing, and grout, can support veneer, protect occupants from the elements, and resist shear and gravity loads. The masonry panel transfers shear to the beams and columns surrounding it.

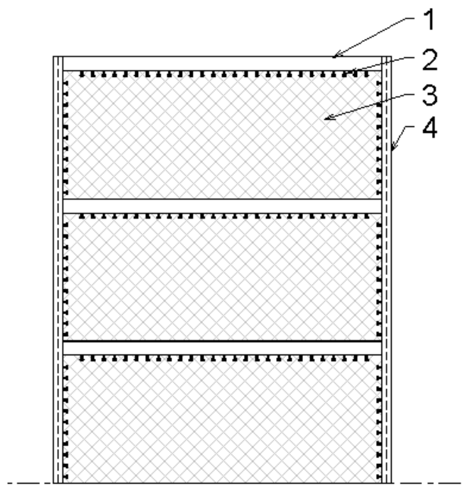


Figure 9 - Type III Hybrid Masonry Elevation

In Type III hybrid masonry, masonry panels distribute loads to the beams above and below the individual panels in one of two ways depending on the reinforcement connection to the beam at the base of the panel. The load transfer method used at the base of the masonry panel depends on whether or not the reinforcement is welded to the beam supporting the panel. These two methods are similar to the two options in Type II hybrid masonry. If the reinforcement is welded to the beams, then tension and compression are used to resolve the overturning moment caused by the lateral loads. A diagram of the force transfer between the panel and the beams utilizing tension is shown in Figure 10. As shown, the tension is opposite the compression at the base of the wall as explained for Types I and II.

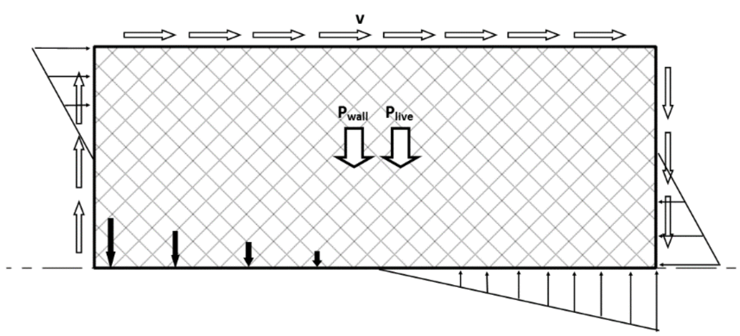


Figure 10 - Type IIIa Hybrid Masonry Panel Free-Body Diagram

The other type of transfer for Type III masonry does not rely on tension in the reinforcement to transfer loads to the beam, so the reinforcement does not need to be welded to the beam supporting the masonry panel. A diagram of load transfer from the panel to the beams and columns is shown in Figure 11, in which compression zones form on the top and base of the panel against the beams at opposite corners.

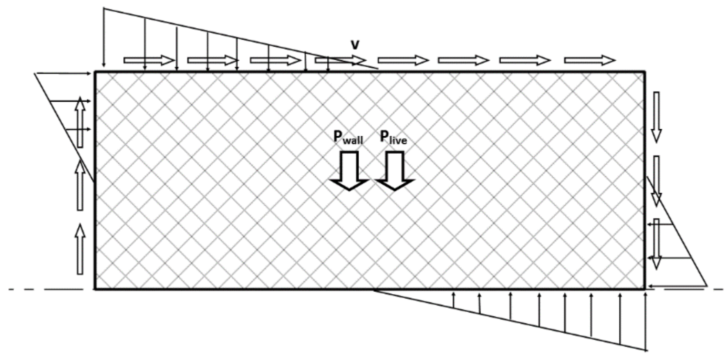


Figure 11 - Type IIIb Hybrid Masonry Panel Free Body Diagram

The addition of shear studs at the columns causes the masonry to transfer shear at the sides of the panel. The masonry is also in direct contact with the columns, also causing compression to transfer at the column locations. Type IIIa and Type IIIb masonry effectively act as confined masonry walls. Because Type III hybrid masonry is complex and little code currently govern its design, it is not discussed further in this research, although many of the basic principles are consistent with other variations of hybrid masonry.

Chapter 3 - Study Parameters

As an emerging structural system, hybrid masonry does not have significant quantitative data to support its advantages. Although hybrid masonry is an unfamiliar concept to apply and study, it relies on familiar design practices. All relevant elements, such as steel gravity framing and masonry shear walls, are commonly designed and used in industry, and those elements have well-developed codes to govern their design and construction. Hybrid masonry panels of Type I and Type II masonry are based on existing codes as non-load-bearing and load-bearing masonry shear walls, respectively. This study was devised to help quantify a hybrid system's advertised advantages. The study compared the advantages of three lateral systems: moment frame with masonry infill, hybrid Type I masonry, and hybrid Type II masonry.

Objectives

Hybrid masonry has been suggested for use as the lateral resisting system of primarily low-rise buildings in low to moderate seismic conditions such as seismic design categories A, B, and C . However, advantages of the hybrid masonry system are primarily supported by conjecture, lacking quantitative data to support the statements. Feasibility of hybrid masonry as a lateral system has been supported by other studies such as Seismic Design and Viability of Hybrid Masonry Builidng Systems by Eidini, Abrams, and Fahnestock and Seismic Design and Viability of Hybrid Masonry with Fuse Connectors by Asselin and use in industry. Familiar concepts and codes are being used to design hybrid masonry systems in the United States already. The concept could gain further acceptance with additional quantitative data to support its advantages. The objective of this parametric study was to investigate the advantages of hybrid masonry quantitatively, such as improved constructability, increased redundancy, and reduced construction costs according to the literature.

The first proposed advantage of hybrid masonry is simplified constructability of framed buildings with masonry infill. Many architects prefer CMU as back up when brick veneer is used on a project Providing the CMU infill between the steel frames interferes with braced frames. This interference increases the difficulty of detailing the connections and placing the masonry infill. Considerable measuring, cutting, fitting, and refitting would be required (Moreels, 2016). Therefore, steel moment frames are commonly used instead of braced frames for the lateral system, but this use significantly increases the costs for each beam-to-column connection. For masonry construction, a practicing masons were interviewed on their thoughts regarding construction of masonry panels. Their comments and specific questions regarding construction requirements for the panels are presented. Difficulties and time required to construct the masonry walls were compared to control infill masonry of the moment frame system based on the rating system, discussed later on in this section thereby providing quantitative data to compare the investigated lateral systems. For steel constructability, the main differences between the systems are required connection types and member sizes. Typical connections were investigated for their constructability based on bolts, welds, and plates required. Weights and dimensions of steel members from the final designs were also compared.

Another proposed advantage of hybrid masonry is the design or increased redundancy of the structural system. Redundancy effectively discourages progressive collapse of a structure The opportunity for progressive collapse occurs when one or more structural elements fail. If the structure remains stable and redistributes the loads, it avoids progressive collapse. Therefore, introducing reinforced, full-height masonry walls in the same plane as the beams would increase the redundancy of the system. Investigations of buildings surrounding the location of the previous World Trade building show the advantages of redundancy in transitional masonry buildings, which are similar to hybrid masonry (Biggs, 2004). For example, if a column fails, the attached beam would likely come to bears on the wall below instead of crashing to the next floor down, saving the structure below from the impact and rearranged load. A system of each lateral type was generated, and one structural element was randomly removed. If a logical load path remains after the removal, the system earns one point. This process was repeated multiple times, and reported as the system’s redundancy score.

The final proposed advantage of hybrid masonry is reduced construction cost. Since construction costs are highly dependent on location, material, and time, material consumption was primarily used to compare the lateral systems. Structural steel and reinforcement were reported by weight in tons, and masonry was reported by whole individual units required. Concrete and grout were reported by cubic yards required, bolts were reported by quantity, and welds were reported by total equivalent length. Equivalent length is used to express the length a welder must move with a single weld pass. Therefore, a weld length that required more than one pass to build the required throat was reported as the actual length times the number of passes required. Items unique to a lateral system, such as hybrid masonry Type I links or shear studs for hybrid masonry Type II, were converted to the most similar main material of steel, masonry, concrete, or connectors.

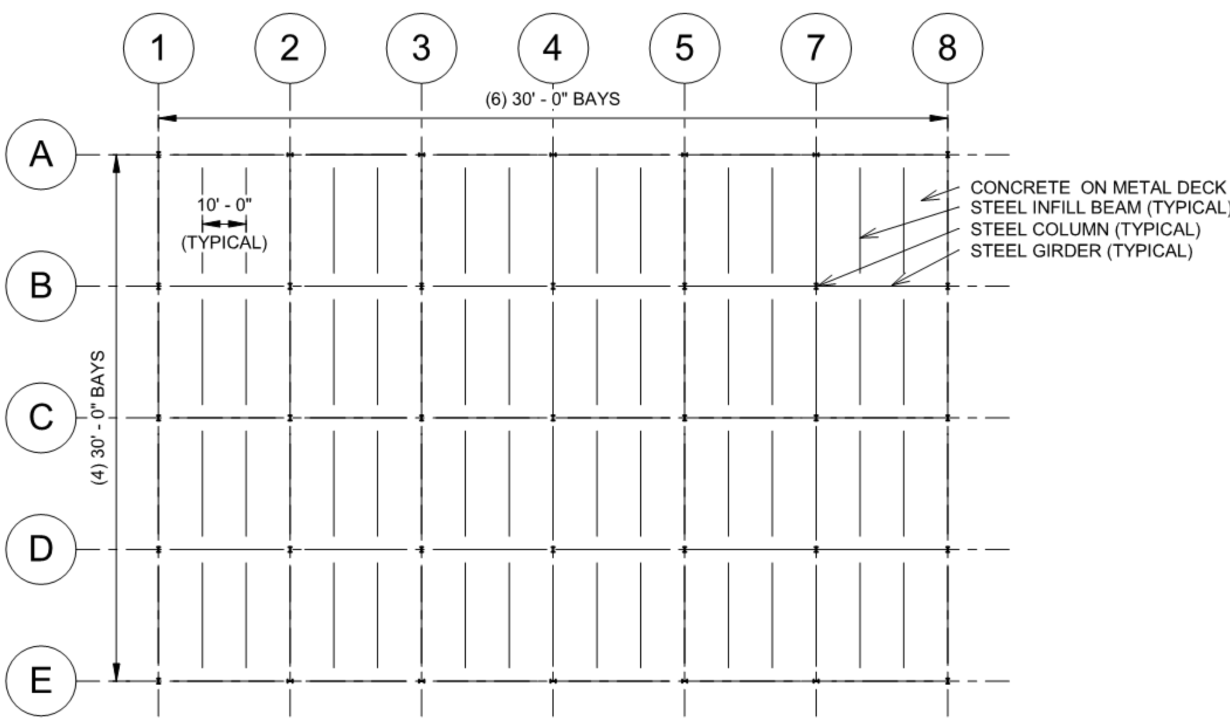
Finally, this study classified each attribute (constructability, design, and economics) into subcategories and inserted them into a matrix to help quantify each system as a whole. The moment frame was used as the control or standard for this decision matrix. Each attribute of the system was compared to of the like attribute of the moment frame system and given a score. Positive values indicated an improvement, and negative values indicated a setback. The score magnitude demonstrated the severity of the change. Because of the small sample size for the study, the magnitudes were limited to one for minor changes, two for significant changes, and three for outstanding changes. Although this method is somewhat subjective, it provided an adequate overall comparison of all three systems.

Project Description

The building for this parametric study is based off of a prototype building created by the SAC Joint Venture which stands for the combined efforts of Structural Engineers Association of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering, The model building was devised to study moment frames after the North Ridge earthquake in 1994 (FEMA 355C, 2000). SAC prototype buildings have been used for various studies because the buildings allow research to start with a similar base as other research, thereby encouraging familiarity with foundation of the study and promoting efficient comparisons of research.

This study investigated a 65,000 sq. ft., three-story, steel frame office building in Kansas City, Missouri. Figures 12 and 13 depict the basic framing layout of the building. The building consisted of 30 ft square bays six bays long in the east-west direction and four bays deep in the north-south direction. Girders spanned the east-west direction, and beams spanned perpendicular to the girders at 10 ft typical spacing. In the figures, thick lines around the perimeter of the building denote the frame layout. The hybrid frames were arranged to keep the center of the building open and available for large openings while maintaining a symmetrical layout to reduce torsion. The floors were comprised of a 3-in. metal deck with 2.5 in. of concrete fill for a total thickness of 5.5 inches. Although the roofs decks in this location are not commonly filled with concrete, this study used the same construction for the roof as the floors in order to maintain similarity to the original SAC prototype building. Concrete filled roof decks are commonly used in high seismic areas where the metal deck alone is not capable of resisting the shear in the roof diaphragm. The roof was flat with a minimum slope of 0.25 in./ft. to discourage ponding. The story heights measured 13 ft. between each level of the building, resulting in an overall building height of 39 ft. with an additional 3.5-ft. parapet, totaling 42.5 ft.

A key parameter in this study was that the exterior of the building had to be a cavity wall system This could have been a requirement from the architect or owner of the project. This stipulation was used because it is common sense that adding masonry to a project for the purpose of using it as shear walls in a hybrid system would not be advantageous. However, changing required standard infill to a hybrid system was proposed to be advantageous. The exterior of the building was a clay brick façade with 8-in. concrete blocks for support. Cavity wall construction as detailed in Figure 1 was assumed.



SCALE: 1/32”=1’-0”

Figure 12 - Typical Framing Plan with Hybrid Masonry

Codes and Standards

The building was designed to the most current design standards at the time of this study. The 2012 International Building Code and codes referenced within governed the building design. Loads for the building were determined using the 2010 version of *Minimum Design Loads for Buildings and Other Structures* published by the American Society of Civil Engineers (ASCE), or ASCE 7-10. Steel design conformed to the Specification for Structural Steel Buildings in the fourteenth edition of the American Institute of Steel Construction (AISC) Manual, or ANSI/AISC 360-10. Masonry was designed based on the 2011 Building Code Requirements and Specification for Masonry Structures, or TMS 402-11 and TMS 602-11. The masonry design also utilized information from technical briefs from the IMI and the National Concrete Masonry Association (NCMA).

Loads

Loads for the test building were similar to loads of the SAC model building, although small changes were made according to updated values in the ASCE 7-10 and technical documents available from manufacturers. The various types of loads are discussed individually in the following sections.

Dead Load

A majority of the dead loads for the building in this study were determined from the values from the SAC model buildings in FEMA 355C, which primarily specified a load rather than a material thereby preventing investigation and possible updates to the load with current standards. Listed components were investigated and confirmed with current code-suggested values and manufacturers' recommendations. For the 3-in. metal deck with 2.5 in. of normal-weight concrete fill, the load for this study was 50 pounds per square foot (psf) according to the *Vulcraft Metal Deck Catalog*. A total of 15 psf, the value from the SAC model building, was used for the weight of the steel framing. This value was somewhat higher , but comparable to the 10 psf calculated for the members used in this building. Since the gravity members were not the main objective for this report, the original SAC model building steel framing weight was used in this study for consistency.

Roof dead loads from the building in the study are shown in Table 2. The dead loads for the floor were identical, with the exception that the roofing material was not included. Dead loads for the floors are shown in Table 3.

Table 2 – Roof Dead Loads

Roofing	7	psf	FEMA 355c
3" Metal Deck w/2.5" NW Conc.	50	psf	Vulcraft catalog
Ceiling	3	psf	FEMA 355c
MEP Allowance	7	psf	FEMA 355c
Steel Structure	15	psf	FEMA 355c
	82	**psf**	

Table 3 - Floor Dead Loads

3" Metal Deck w/2.5" N.W. Conc.	50	psf	Vulcraft catalog
Ceiling	3	psf	FEMA 355c
Flooring	3	psf	FEMA 355c
MEP Allowance	7	psf	FEMA 355c
Steel Structure	15	psf	FEMA 355c
	78	**psf**	

According to the cavity wall requirement for this study, the exterior walls were standard modular clay brick with a CMU backup. Two main types of these walls were used within the project. The first is for the walls in the hybrid masonry sections. Walls in the hybrid sections required additional grouting and were therefore significantly heavier than their counterparts in the non-hybrid sections. Typical masonry infill sections used CMU that was grouted and reinforced at 48 in. on center because that is the maximum spacing allowed by code and that spacing adequately resisted the out or plane loads due to wind (Masonry Standards Joint Comittee, 2011). Tables 4 and 5 summarize the dead loads retrieved from ASCE 7-10, Table C3-1 for partially grouted and fully grouted panels, respectively.

Table 4 – Masonry Infill Dead Load for Partially Grouted Panel

8" NW CMU grouted 48" OC	44	psf	ASCE 7-10 TC3-1
4" Clay Masonry	39	psf	ASCE 7-10 TC3-1
	83	**psf**	

Table 5 - Masonry Infill Dead Load for Fully Grouted Panel

8" NW CMU, reinf. -solid grout	81	psf	ASCE 7-10 TC3-1
4" Clay Masonry	39	psf	ASCE 7-10 TC3-1
	120	**psf**	

Live Loads

The building in this study was an office building, which should have a typical load of 50 psf according to Table 4-1 in ASCE 7-10. Code also mandates that an additional 15 psf be added to this live load to account for the addition of partitions within the office space (ASCE 7-10 Section 4.2.2), resulting in a live load of 65 psf. An office space also typically contains corridors, but the SAC model building neglected the corridor spaces that would have contributed a load of 80 psf. Neglecting this load is not advisable for design. Although the difference between the two loads is only 15 psf, the corridor spaces should be accounted for in the design. However, because the corridor spaces were not outlined in the building, a blanket load of 80 psf was conservatively used throughout the building in this study. The live load or construction load for the roof or floors was 20 psf according to ASCE 7-10, Table 4-1.

Snow Loads

Detailed investigation of snow loading was deemed unnecessary for the objective of this study because it would primarily add excessive assumptions to the project without contributing significant knowledge. Therefore, for this study, the assumption was made that the construction live load governed any snow load on the roof, thereby allowing comparisons of this research to future research that utilizes the same SAC model building..

Wind Loads

The main objective of this study was to investigate hybrid masonry under seismic loading. Therefore, the assumption was made that wind does not govern for the main lateral force resisting system. Determination of the base shear for wind and seismic load cases in this location confirmed this assumption. Components and cladding loading was found to confirm that the reinforcement spacing in the infill sections of masonry allowed adequate determination of the dead load. Both of wind calculations are available in Appendix B.

Seismic Loads

Seismic load was the controlling load for this study as shown in Appendix A and B. Equivalent lateral force procedure, which was used to design the systems, is a life safety-based design that converts dynamic or moving loads from earthquakes loads applied statically to the structure. The building may not be usable after a significant seismic event, but the structure remains stable to protect occupants (ASCE/SEI 7-10, 2010). An alternative design method is a performance-based design which is not necessary for this study. Many values and coefficients must be collected in order to determine equivalent static seismic force. First, site accelerations must be determined, which require several assumptions. The first assumption is the site class. Site class D, or stiff soil, was assumed for this project because it is the base assumption when soil conditions are not known in enough detail to determine the actual site class (IBC 1613.3.2., 2012). The next assumption was the exact location of the building. The building for this study was located in Kansas City, Missouri to illustrate hybrid masonry lateral force resisting systems in a moderately low seismic location. The exact location for spectral accelerations was the center of the city, or 39.0811°N, 94.56383°W. The site class, location, and importance factor used to calculate the accelerations were found using the United States Geological Survey (USGS) design maps tool. Results showed that based on the buildings location, site class, and importance category the project should be designed for the seismic design category B and gave the spectral accelerations.

The final crucial assumption for the determination of seismic load is the response modification factor for the hybrid masonry system. To date, not enough adequate testing has been conducted on full-scale systems in order to present a response modification factor specific to hybrid masonry. Investigation of the system is underway at the University of Illinois at Urbana-Champaign and the University of Hawaii. Until that research is completed, two main approaches for the response modification factors exist. The first option is to assume a response modification factor between 5 and 7 based on similar ductile combinations of masonry and steel force resisting systems (Asselin, 2013). The second approach is to use the response modification factor based on reinforced masonry walls as suggested in technical documents from IMI and NCMA. Response modification factors for this approach range from 2 to 5 depending on the masonry classification (ordinary, intermediate, or special).

Comparing the two assumptions the dual system approach would result in a base shear of approximately 130–180 kips depending on the range of the response modification factor. Using the masonry approach, the resulting base shear would be approximately 450 kips for ordinary masonry to 180 kips for special masonry. Although the masonry approach results in a significantly larger load than the dual approach, it is documented in multiple locations and is the more conservative approach; therefore, the masonry approach with ordinary masonry was used for this study. Ordinary masonry has a response modification factor of 2. This approach is more conservative because the masonry response modification factor indicates system flexibility, and an ordinary masonry shear wall is less ductile than systems that use steel. A brittle system earns a lower response modification factor which has an inverse effect on the seismic base shear. The seismic response coefficient *Cs*, is the design spectral response acceleration *Sds*, times the seismic importance factor *Ie*, and divided by the response modification factor *R*, as shown in equation 3-2. As the response modification factor decreases, the seismic response coefficient increases. The base shear V, also increases because it is determined by multiplying the effective seismic weight of the building *W*, by the seismic response modification factor as shown in equation 3-1. An increase in the seismic response coefficient requires that the building be designed to higher loads. Therefore, the structure will be pushed less into the inelastic range. Figure 13 graphically explains the elastic and inelastic ranges of design.

$$V = C_s W \qquad \text{(EQ. 3-1)}$$

$$C_s = \frac{S_{DS}}{R/I_e} \qquad \text{(EQ. 3-2)}$$

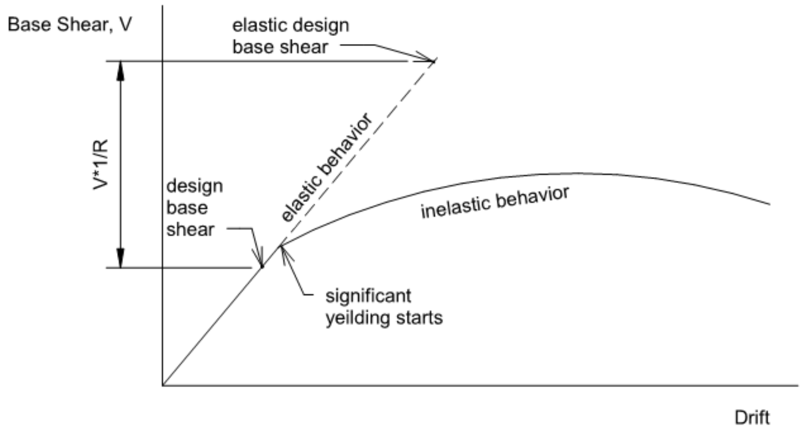


Figure 13 - Inelastic Force Deformation Curve (adapted from SEOAC, 2008)

The linear portion of the curve is the elastic behavior. Elastic design shear is the unmodified shear force due to the mass of the structure and the acceleration at the top of the line. If the structure was to avoid permanent deformations the lateral system would have to be designed for this base shear (SEOAC Seismology Committee, 2008). However, with equivalent lateral force inelastic deformation is used to diffuse energy. The response modification factor, based on available inelastic deformation, allows the design base shear to be reduced. The larger the response modification factor the smaller the design shear.

The exact response modification factor for the project had to be chosen and kept consistent. For typical systems, an ordinary moment frame system has a response modification factor of 3.5, resulting in a 245 kip base shear. Because the hybrid masonry portion of this project utilized the masonry approach, the response modification factor was 2, which corresponded to ordinary reinforced masonry. This system had no height restrictions in seismic design category B (ASCE/SEI 7-10, 2010). The resulting base shear for the hybrid system was 447 kips.

The loads were calculated using ASCE 7-10 with the equivalent lateral force procedure using the described assumptions. A detailed account of the calculations is provided in Appendix A, but the basic process follows. The previous assumptions, the spectral accelerations, building geometry, the period of the building and site, the response modification factor, and the importance category were used to determine the seismic response coefficient. The coefficient was multiplied by the effective seismic weight of the building as defined in Section 12.7.2 of

ASCE 7-10. The resulting product was the base shear for the entire building. That shear was then distributed vertically to each level by considering the proportions of the weight at each floor compared to the total building weight. The shear at each storywas then distributed to the lateral force resisting systems using statics. Since the levels contained more than 2 in. of concrete, they were assumed to act as rigid diaphragms, and accidental torsion also had to be added to the seismic load for each frame. The summation of the primary shear and shear from accidental torsion was the load the individual frames were designed to resist.

Process

In this study, the described loads were used to design three lateral force resisting systems:

1. An ordinary moment frame system with masonry infill with a response modification factor of 3
2. A Type I hybrid masonry system with a response modification factor of 2 for ordinary masonry
3. A Type II hybrid masonry system with a response modification factor of 2 for ordinary masonry

All three lateral force resisting systems were designed according to the same principles. Each system was initially designed to resist forces from gravity. Next, the gravity design was redesigned to resist additional lateral forces. Finally, the system was checked for serviceability and adjusted as required. Unique loading combinations were required for the design of each of these stages.

Design Load Combinations

The load factor resistance design (LRFD), or strength-based design, was used to design the systems in this study. First the loads were factored using load combinations from ASCE 7-10, Sections 2.3.2 and 12.4.2.3. These load combinations are shown in equations 3-3 through 3-7. Wind combinations were omitted because the focus of the study was seismic design. The load variables are defined as follows: design spectral response acceleration *Sds*, dead load *D*,

$$1.4D \qquad \text{(EQ 3-3)}$$

$$1.2D + 1.6L + .5L_r \qquad \text{(EQ 3-4)}$$

$$1.2D + 1.6L_r + .5L \qquad \text{(EQ 3-5)}$$

$$(1.2 + .2S_{DS})D + \rho Q_E + L \qquad \text{(EQ 3-6)}$$

$$(.9 - .2\,S_{DS})D + \rho Q_E \qquad \text{(EQ 3-7)}$$

redundancy factor ρ, seismic force Q_E, live load L, and roof live load Lr. Since the building was in design category B, the redundancy factor was equal to 1.

Moment Frame Design Process

Moment frames were designed using familiar methods and conventions for ordinary moment frames according to AISC 360-10 and AISC 341-10. Ordinary moment frames with bolted connections were used for this system. The basic design flow started with designing the beams, girders, and columns for gravity loads. Second, the members were designed to resist lateral loads based on strength. Next, the steel moment frames were designed for drift. The drift limit, discussed in the serviceability section, governed the design of the moment frames. The system was originally designed with two bays per side to mimic the hybrid masonry systems. However, the member sizes became ridiculously large to control drift at the first floor, so the bay count was increased to control drift and the previous checks were redone with the new bay count. Four moment frame bays per side allowed deflection to be controlled and prevented any column from having to be designed for biaxial bending. The resulting layout is depicted in Figure 14. Specific design steps for moment frames were not the focus of this research and are not discussed in this report. Step-by-step design examples for ordinary moment frame design are provided in the AISC Seismic Design manual.

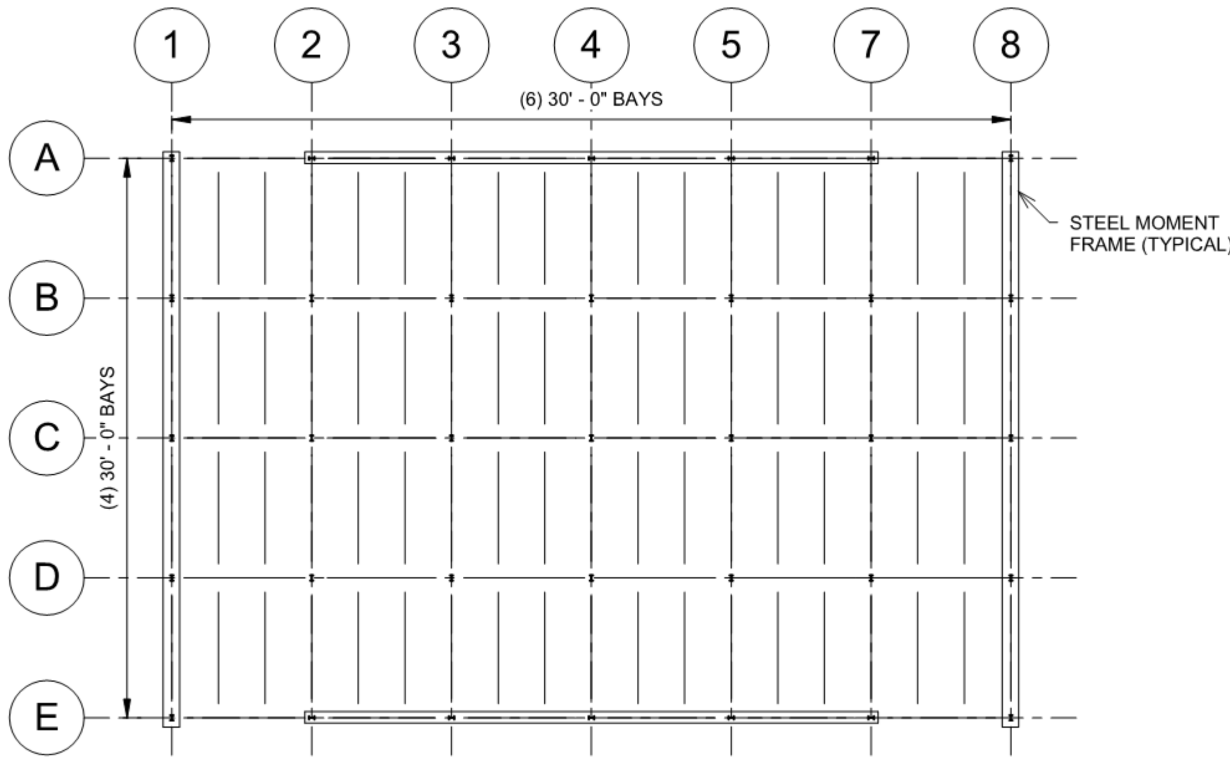


Figure 14 - Moment Frame Layout

Type I Hybrid Masonry Design Process

As discussed, hybrid masonry is composed of three main portions the steel frame, connectors, and a reinforced masonry panel. The following sections describe each element, and a sample design for a Type I hybrid masonry frame is provided in Appendix B.

Steel Frame

Steel-frame members must consider all LRFD load conditions listed in the load combination section, equations 3-3 through 3-7, during design. Columns and beams were initially designed for the full gravity load. For design of the columns, axial load was the controlling factor, specifically additional load due to overturning of the frames, as illustrated in equation 3-8. Figure 14 shows a basic free-body diagram with applied seismic loads and base reactions for a single frame.

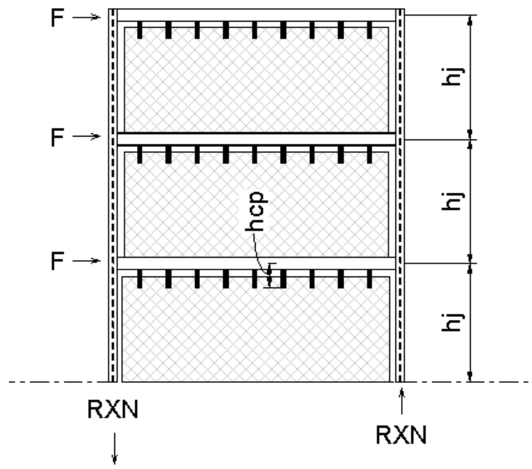


Figure 15 - Type I Hybrid Masonry Free-Body Diagram

Figure 15 can also be used to describe the calculation of the axial loads within the columns. Equation 3-8 simplifies calculations of the axial force due to lateral loads (Eidini, Abrams, & Fahnestock, 2013).

$$P_i = \frac{\sum_{j=i+1}^{n}\left(F_j \times h_j\right) + F_i \times h_{cp}}{B} \quad \text{(EQ. 3-8)}$$

In this equation, the *j* subscripts represent levels above the level in question, and the subscript *i* indicates the level in question, *F* is shear force due to seismic loading at a particular level, *B* is the distance between the columns, *h* is the story height of that level, and h_{cp} is the distance between the center of the beam and the connection to the masonry, or the eccentricity, of the connecter plate assuming that this distance is consistent across the levels. The result is the axial load *P* on the level in question caused by the lateral forces. For the hybrid masonry portions of the parametric study the story heights were 13 ft at each floor and the distance between the columns was 30 ft. For Type I hybrid masonry the eccentricity was the sum of one half of the beam depth, the gap between the beam and the top of the masonry, and the required edge distance of the through bolt; approximately 3 ft. The resulting seismic axial load on the column was approximately 48 kips.

The shear, flexure, stability, and deflection were checked for the beams in the frames at each level. Since the exterior beams and girders supported masonry, deflection for the beams and girders were limited significantly. However, deflection governed the design in all beam cases for this system. Applicable limits are discussed in the following serviceability section of this report. Beams that support hybrid masonry must also be designed for seismic load transferred from masonry panels to the beam that supports the panel, as previously described in the description of Type I hybrid masonry. Sample shear and moment diagrams from these loads are shown in Figures 16 and 17.

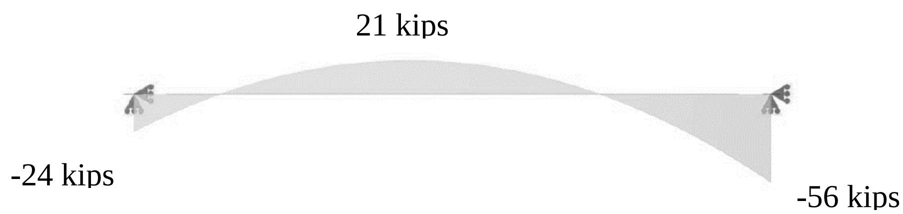


Figure 16 - Type I Hybrid Masonry Shear Diagram

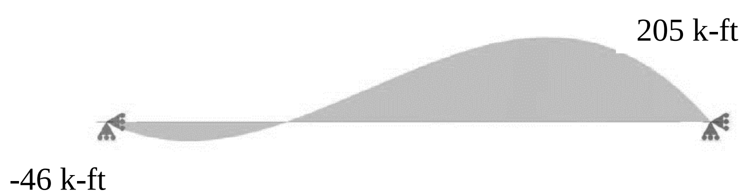


Figure 17 - Type I Hybrid Masonry Moment Diagram

The shear and moment diagrams in Figures 15 and 16 do not resemble a typical floor framing beam with uniform load because lateral load transferred from the shear panel skewed the forces. In these diagrams the seismic load points to the right, causing the masonry panel to press down at the right side of the beam and lift up at the left. This action increases the shear on the right side of the beam, decreases the shear to the left, and skews the maximum moment to the right side of the beam.

Connections between the beams and columns are only required to be typical shear type connections because masonry panels are expected to resist all of the lateral shear force in Type I hybrid masonry due to the large difference in stiffness between the steel frame and the masonry panel (Eidini, Abrams, & Fahnestock, 2013). The rigid stiffness of the masonry panel attracts the lateral load instead of the less rigid steel framing. The only seismic forces within the steel framing is the shear as it pass through the beam from the diaphragm to the masonry and axial chord forces in the columns.

Hybrid Connecters

Connections between the steel frame and the masonry panels are a key component to a Type I hybrid masonry system. As described, two kinds of links may be used for this system: the ductile fuse or the link plate. Since yielding is concentrated in the masonry for Type II hybrid masonry, the yielding component was kept consistent for this study. Link plates were chosen to transfer all loads to the masonry wall where masonry crushing and yielding of the reinforcement provided ductility for the system. The link plate used for the study was S-P6_T4-01 from Ozaki-Train, Johnson, and Robertson (2011). The researchers theoretically calculated link strength and compared it to experimental results. The first yield capacity of a pair of links was 14 kips with an ultimate capacity of 30 kips. A detail for this link is provided in Figure 18. The failure mechanism for the links was yielding around the bolts through the masonry with eventual rupture at this location (Ozaki-Train, Johnson, & Robertson, 2011). To ensure this failure mechanism, the link must be protected from volatile failures such as masonry breakout, masonry crushing, anchor bolt pry-out, and steel yielding (Eidini, Abrams, & Fahnestock, 2013). The main way to guard against masonry type anchor failures is to follow geometry-based guidelines in TMS 402-11. More detailed information on the performance of link connections is available in the report by Ozaki-Train, Johnson, & Robertson (2011) and recent reports from the University of Hawaii.

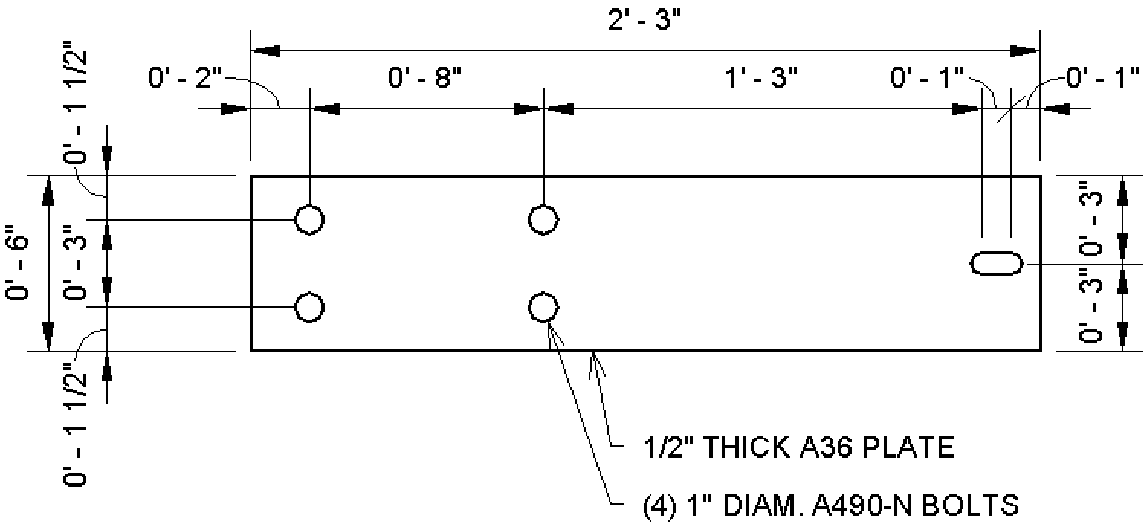


Figure 18 - Hybrid Masonry Link Connection (adapted from Ozaki-Train, Johnson, & Robertson, 2011)

The link was the determining factor for the quantity of Type I hybrid frames required for the building. Total base shear was divided by the yielding capacity of a single pair of links to determine the total number of required links. A small safety factor of approximately 1.5 was applied to ensure that the links did not fail prematurely. The number of required links was then divided by the available space in one of the masonry panels. The available space was rationalized as one pair of links in every other cell of the masonry panel, meeting all requirements to avoid volatile failures. Based on a base shear of 447 kips, a link yield capacity of 14 kips, and 30-ft. bays, the structure required four frames of Type I hybrid masonry to resist lateral seismic load in each direction. The frame layout is shown in Figure 19.

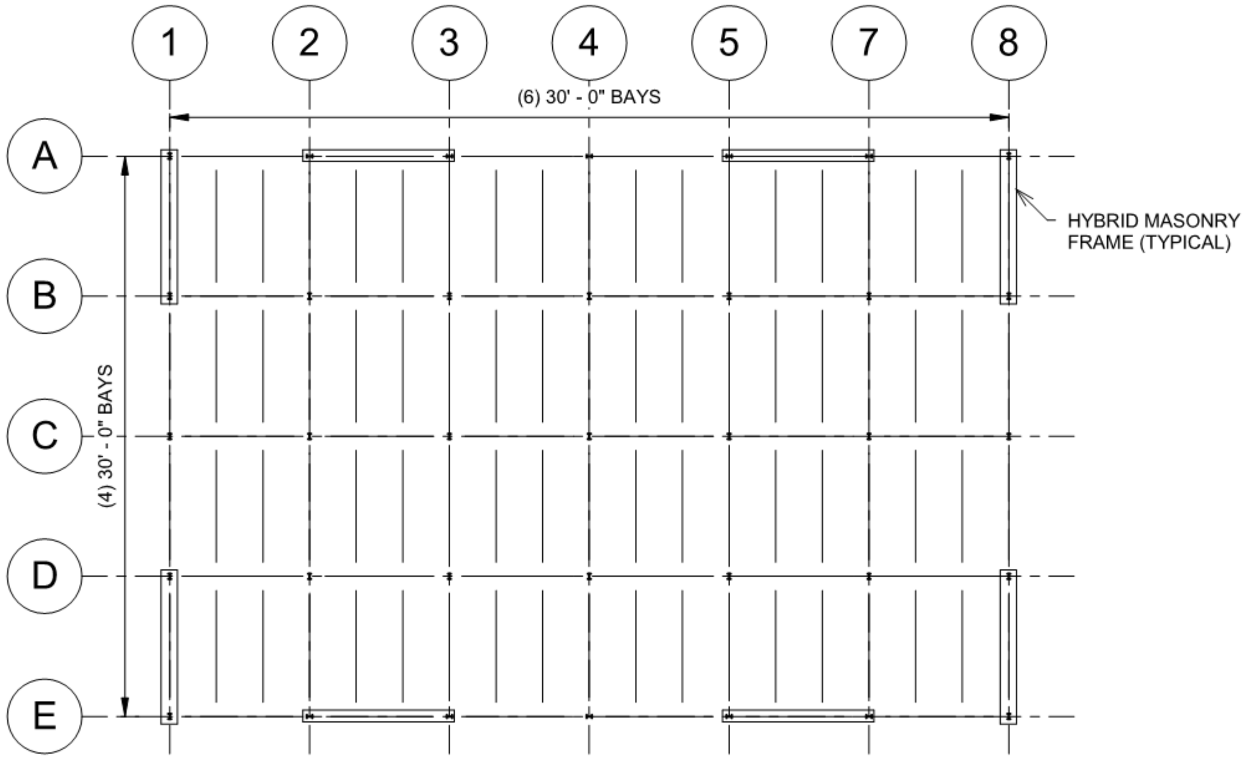


Figure 19 - Type I Hybrid Masonry Frame Layout

Masonry Panel

For design, a hybrid masonry panel can be considered to be a non-load-bearing masonry shear wall. Design of a non-load-bearing shear wall is well-defined in the parameters of the Building Code Requirements for Masonry Structures (Masonry Standards Joint Comittee, 2011). Masonry walls must be checked regarding axial strength, shear strength, and flexural strength. Although masonry can be designed with allowable stress design or strength design, this study used strength design to maintain consistency. For strength design, loads on the panel must be factored using the LRFD load combinations shown as equations 3-3 through 3-7 in this report. For Type I hybrid masonry, axial strength does not control since the panel is not load bearing; therefore, this check was confirmed by inspection. The next check is a shear check. Masonry shear strength is a combination of the strength of the masonry and the reinforcing steel. Masonry shear strength *Vnm* is dependent on the ultimate moment to shear ratio *Mu*/(*Vudv*), the cross-sectional area of the masonry *Anv*, the masonry compressive strength *f'm*, and the ultimate axial load *Pu*, according to equation 3-9. The reinforcing steel strength *Vns* is one-half the area of a vertical steel bar *Av* times the yield strength *fy* and shear depth *dv* divided by the spacing of the steel *s*, as shown in equation 3-10.

$$V_{nm} = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_{nv}\sqrt{f'm} + .25P_u \qquad \text{(EQ. 3-9)}$$

$$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v \qquad \text{(EQ. 3-10)}$$

The shear panel must be designed so that the nominal shear capacity does not exceed a maximum value based on the ultimate moment-to-shear ratio. The limits are defined by equations 3-11 and 3-12. For values between the two equations interpolation is required.

$$if\ \frac{M_u}{V_u d_v} \leq .25\ then\ \ V_n \leq 6A_{nv}\sqrt{f'm} \qquad \text{(EQ. 3-11)}$$

$$if\ \frac{M_u}{V_u d_v} \geq 1\ then\ \ V_n \leq 4A_{nv}\sqrt{f'm} \qquad \text{(EQ. 3-12)}$$

The final check for the masonry wall is flexural capacity, or overturning capacity. Flexural strength may be checked based on the creation of an interaction diagram or using statics to compute the strength based on the cross section of the entire masonry panel. Both methods use the same general theories and require determination of the neutral axis *c*. The interaction diagram balances the ultimate axial force with available compression and tension forces in the masonry panel, as described in equation 3-13. Compression forces *C* in the masonry were determined using Whitney's stress block to approximate the area, thickness *t* times 80 percent of the distance to the neutral axis, of the masonry affected by psuedo uniform stress in the compression zone of the masonry. This area was multiplied by 80 percent of the masonry strength per equation 3-14. Tension *T* was determined by multiplying the strain in the reinforcement by the steel yield strength.

$$\sum Fy = 0, therefore \qquad \frac{P_u}{\emptyset} = C - \sum T \qquad \text{(EQ. 3-13)}$$

$$C = 0.8f'm0.8ct \qquad \text{(EQ. 3-14)}$$

Unique considerations for the masonry panel used in a hybrid system included the connection to the beam, which must be capable of transferring compression and tension from the masonry wall through the slab to the beam. The masonry of the wall and the concrete of the slab were strong in compression; the compression was transferred through direct contact from the

CMU wall to the slab and to the beam below. In order to transfer the tension, the rebar within the reinforced masonry must be designed to meet tension requirements of the panel and must be secured to the beam. The rebar must continue from the panel through the slab to be welded to the beam (IMI Technology Brief 02.13.02, 2010). An illustration for this connection is shown in Figure 19.

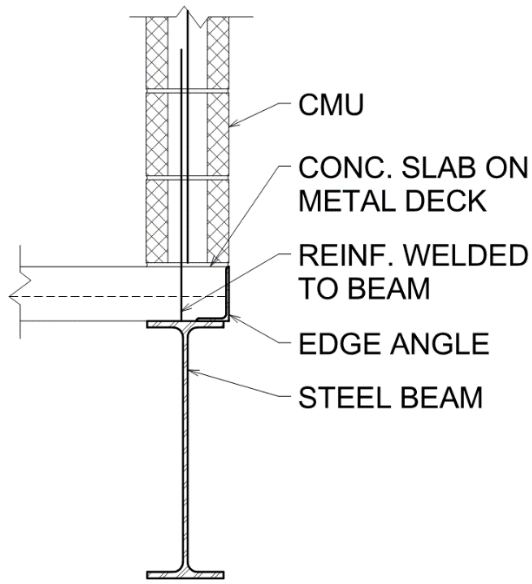


Figure 20 - Type I Hybrid Masonry Panel Base Connection

Type II Hybrid Masonry Design Process

Type II hybrid masonry is also comprised of three main parts the steel framing, connectors, and a masonry panel. Each portion is discussed in the following sections, and a sample design for a Type II hybrid masonry frame is provided in Appendix C.

Steel Framing

Design of the steel framing for Type II is similar to the design of Type I framing. The most significant difference is that Type II masonry participates in load sharing of gravity loads between the frame and the masonry walls load sharing is possible because there is no gap separating the top of a masonry panel and the bottom of the beam. Load sharing allows the beam to be sized only for its self-weight, the dead load of the related floor and framing, and a construction load similar to composite steel design. To ensure the designed load sharing, the masonry must be constructed from the base of the structure to the top. The masonry wall panel on the level below must achieve adequate strength to support the panel on the floor above before it can be constructed. The masonry wall would then be able to accept the additional load, as discussed in the masonry panel section of this document. For the columns, additional load related to overturning was still considered; the same equation as in Type I, equation 3-8, was used for additional load for Type II columns. The only difference in the load calculation is that shear studs were used for connections between the underside of the beam and the top of the masonry panel; therefore, the eccentricity of the connection decreased considerably, reducing the h_{cp} value in equation 3-8. For the hybrid masonry portions of the parametric study the story heights were 13 ft at each floor and the distance between the columns was 30 ft. The resulting seismic axial load on the column was approximately 46 kips.

Beams are typically designed for shear, flexure, stability, and deflection. Since exterior beams and girders in the Type II hybrid system support less load than similar beams in the other two systems and are supported continuously from underneath, once the wall is in place, deflection of the beam is no longer a concern. Deflection is another point that confirms the importance of correct sequencing of construction of the Type II masonry panels. Beams that support hybrid masonry must be designed for seismic load transferred from the masonry panels to the beam. In Type II hybrid masonry, the beam resists compression loads from the panel directly above and below the beam, as described previously in the document. The beam, however, is supported from above and below by a masonry panel. Local buckling of the beam web and beam-to-column connections were designed for incidental seismic loads.

Connections between the beams and columns for Type II hybrid masonry need only be typical shear tab connections because the masonry panels are expected to resist nearly all lateral forces. This expectation is due to the large difference in stiffness between the steel frame and the masonry panel the same as for Type I hybrid masonry (Eidini, Abrams, & Fahnestock, 2013).

Hybrid Connecters

As mentioned, connectors for Type II masonry are typical shear studs. The quantity of studs required to transfer the shear load from the beam to the wall are distributed throughout the underside of the beam. The quantity was found by determining the maximum shear and dividing by the capacity of a single shear stud. The maximum capacity is limited by the concrete strength, or the shear strength, of the stud, as shown by equation 3-15 from AISC 360-10, which describes the strength of the shear stud based on the cross sectional area of the stud *Asc*, concrete strength *f'c*, and concrete modulus of elasticity *Ec*, with a limit based on the ultimate steel strength of the stud. The ultimate strength of the stud is determined by multiplying the area of the stud by the tensile strength *Fu,* and adjustment factors *Rg* and *Rp*. These adjustment factors are based on the geometry of the connection and the interaction between the metal deck, steel member, and stud. For the system in this study, the studs were welded directly to the member, resulting in adjustment factors of 1.0 and 0.75 respectively. The masonry panel was then constructed flush to the bottom of the beam, allowing transfer of the vertical forces as well.

$$Q_n = \min(\frac{A_{sc} * sqrt(f'_c E_c)}{2}, R_g R_p A_{sc} F_u) \qquad \text{(EQ. 3-15)}$$

The quantity of shear studs that was required governed the number of Type II hybrid masonry frames required. Four frames were required per direction similar to the Type I hybrid masonry. They frame layout is shown in Figure 21.

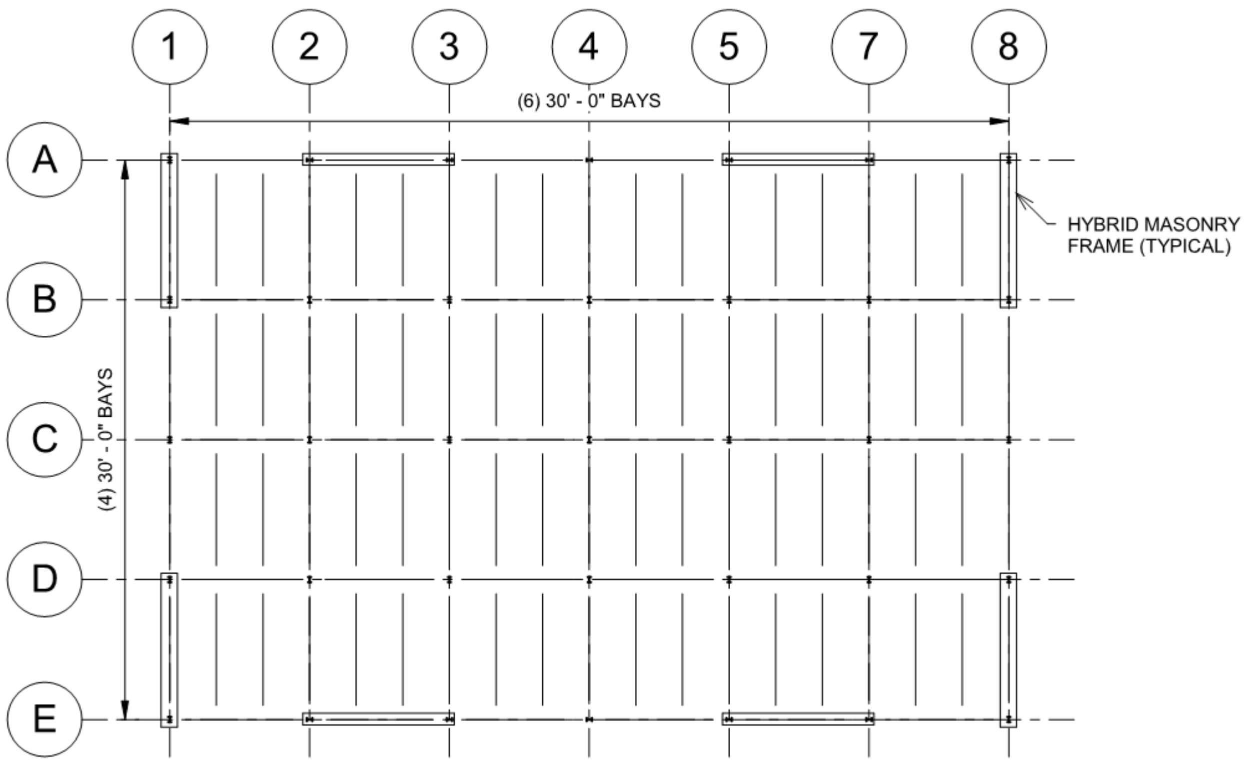


Figure 21 - Type II Hybrid Masonry Frame Layout

Masonry Panel

A Type II hybrid masonry panel can effectively be considered to be a load-bearing masonry shear wall. Design of a load-bearing shear wall is well-defined in parameters of the *Building Code Requirements for Masonry Structures* (Masonry Standards Joint Comittee, 2011).

Identical to Type I hybrid masonry, masonry walls for Type II must be checked in regards to axial strength, shear strength, and flexural strength. All the same assumptions, limit states, and design criteria that apply to Type I also apply to Type II hybrid masonry, with the exception that axial strength should be directly checked. The first step in determining axial capacity of the panel is to determine slenderness of the panel. Slenderness is defined as the height of the wall *h*, over the radius of gyration *r*. The radius of gyration is the square root of the quantity of the moment of inertia divided by the area of the wall. With simplification, the slenderness can be expressed as equation 3-16, where *h* represents wall height and *l* represents wall length. Calculated slenderness determined by how much the nominal axial strength *Pn*, of the wall was reduced. The strength of the wall is defined as 80 percent of the compressive strength of the masonry, masonry area *Am* times masonry compressive strength *f'm* and the compressive strength of the steel, area of steel *Ast*, times the yield stress of the reinforcing steel *fy*. The resulting strength is reduced by a factor for slenderness according to equations 3-17 and 3-18.

$$slenderness, \frac{h}{r} = h/\sqrt{\frac{l\text{^}2}{12}} \qquad \text{(EQ. 3-16)}$$

$$if\,\frac{h}{r} \leq 99\ then\ P_n = 0.8(.08f'm(A_m - A_{st}) + f_y A_{st})(1 - \left(\frac{h}{(140r)}\right)^2) \qquad \text{(EQ. 3-17)}$$

$$if\,\frac{h}{r} > 99\ then\ P_n = 0.8(.08f'm(A_m - A_{st}) + f_y A_{st})(1 - \left(\frac{70r}{h}\right)^2) \qquad \text{(EQ. 3-18)}$$

The remainder of the required design checks for a Type II hybrid masonry panel are stated in the section on Type I hybrid masonry panel design.

Serviceability

Horizontal and vertical deflection was examined for each of the three lateral systems in question. Horizontal deflection, or drift, of the systems must be maintained under certain limits for structural stability. Seismic drift was computed at strength levels using the combinations in equations 3-7 and 3-8 discussed previously. For drift, the frame must not exceed the maximum stability coefficient θ, or the maximum allowable story drift. The stability coefficient is found using equation 3-19. found in ASCE 7-10, Section 12.8 where *Px* is the total vertical design load at and above the level in question; *Ie* is the seismic importance factor, *Vx* is the seismic story shear at the level in question; h_{sx} is the story height below the level in question; C_d is the deflection amplification factor per ASCE 7-10 Table 12-2.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \tag{EQ. 3-19}$$

The stability coefficient is a measure of the angle of rotation in radians for the relevant story. The maximum allowed rotation is defined by equation 3-20 per ASCE 7-10, Section 12.8. Beta is the ratio of shear demand to shear capacity and is generally conservatively taken as 1.

$$\theta_{max} = \frac{.5}{\beta C_d} \leq 0.25 \tag{EQ. 3-20}$$

Allowable story drift is defined by Table 12.12-1 in ASCE 7-10. A modified table showing only importance category II limits is shown in Table 6

Table 6 - Drift Limitations (ASCE 7-10)

Structure	Allowable Drift Δa
Structures other than masonry shear wall structures (with exterior walls designed to accommodate story drifts)	$0.025h_{sx}$
Cantilevered Masonry Shear Walls	$0.010h_{sx}$
Other Masonry Shear Walls	$0.007h_{sx}$
All Other Structures	$0.020h_{sx}$

Because the structure in this study was clad in masonry veneer, the assumption was made that the exterior walls were not designed to accommodate significant drift, thereby eliminating the least stringent limit for all lateral systems investigated. Two categories of limits remained: masonry shear walls and other. Masonry is more rigid and brittle than other lateral systems, so masonry shear walls have a stricter limit for story drift. To protect the masonry panels in this study's hybrid systems, a factor of 0.007 of the story height was used as the drift limit.. In the moment frame condition, the masonry was considered sacrificial, so a factor of 0.02 of the story height was used as the limit for the moment frame system. Consistent story heights of 13 ft. set the drift limit just over 1 in. for the masonry shear walls and just over 3 in. for the moment frame system.

Vertical deflections were also critical to all investigated systems, especially in cases where the beams of each system supported brittle masonry. Suggested deflection limits for masonry supported on a steel beam are provided in AISC's *Design Guide 3*. This condition applies to the moment frame system and the Type I hybrid masonry system. The limit for dead load is smallest of 1/600 of the span length or 0.375 in. The limit for live loads is the smaller of the span divided by 360 and 0.25 in. For a combined dead and live load, the limit is the smaller of the span divided by 480 and 0.625 in. (AISC Design Guide 3, 2012). With a 30-ft. span, as in this project, governing deflection limits were 0.375 in., 0.25 in., and 0.625 in., respectively. Less stringent typical floor and roof member deflections apply to Type II hybrid masonry beams because the beams do not carry the load of the masonry; they only carry the dead load of the floor and a construction live load. For floor member live loads, the limit is the smaller of the span divided by 360 and 0.75 in. For a combined dead and live load, the limit is the smaller of the span divided by 480 (IBC, 2012). Deflection limits were checked at stress levels which means the actual loads the structure may experience instead of loads increased by factors like in strength design. The applicable load combinations for stress levels in this project were from 7ASCE 7-10, Section 2.4.1 and 12.4.2.3 as listed in equation 3-21 through 3-27. The load variables are defined as follows: design spectral response acceleration *Sds*, dead load *D*, redundancy factor ρ, seismic force Q_E, live load *L*, and roof live load *Lr*. Since the building was in design category B, the redundancy factor was equal to 1.

$$D \qquad \text{(EQ. 3-21)}$$

$$D + L \qquad \text{(EQ. 3-22)}$$

$$D + L_r \qquad \text{(EQ. 3-23)}$$

$$D + .75L + .75L_r \qquad \text{(EQ. 3-24)}$$

$$(1.0 + .14S_{DS})D + .7\rho Q_E + L \qquad \text{(EQ. 3-25)}$$

$$(1.0 + .10S_{DS})D + .525\rho Q_E + .75L \qquad \text{(EQ. 3-26)}$$

$$(.6 - .14S_{DS})D + .7\rho Q_E \qquad \text{(EQ. 3-27)}$$

Chapter 4 - Design Comparison

At the completion of the study three lateral systems were designed for a three-story office building with a cavity wall supported by steel framing: moment frame with masonry infill, Type I hybrid masonry, and Type II hybrid masonry. The moment frame with masonry infill was used as the control, or standard, option to compare with new hybrid masonry options. The resulting designs and descriptions as to how the systems faired in constructability, lateral stiffness, redundancy, and economics are presented in this chapter.

Moment Frame Design

The control system, or moment frame system design, included structural steel, masonry infill, and fixed beam connections. The quantity and layout of bays was governed by drift. The system was originally designed with two bays per side as in each of the hybrid masonry systems. However, the member sizes became ridiculously large to control drift at the first floor, so the bay count was increased to control drift. Another possible solution would have been to fix the base of the columns, which would have dramatically increased the size of the foundation and complexity of the column anchorage to the foundation. Increasing the bay count allowed the foundation and base plates of the moment frame system to stay comparable to the hybrid systems. Four moment frame bays per side allowed deflection to be controlled and prevented any column from having to be designed for biaxial bending. The typical moment frame bay consisted of W27x129 columns with W24x62 beams at each floor and W21x55 beams at the roof as shown in Figure 22. Steel sizes were based on controlling drift so that they applied to the beam and girder cases. The concrete masonry infill was merely provided for out-of-plane loads, so it only required minimum reinforcement of No. 5 rebar at 48-in. spacing. The moment connection for this project was an endplate connection that eliminated field welding. The connection had a 1.5-in. endplate, beam stiffeners, transverse stiffeners, doubler plate, 16 bolts, and 9/16 inch full penetration weld around a majority of the beam profile. Six moment connections were required for each bay.

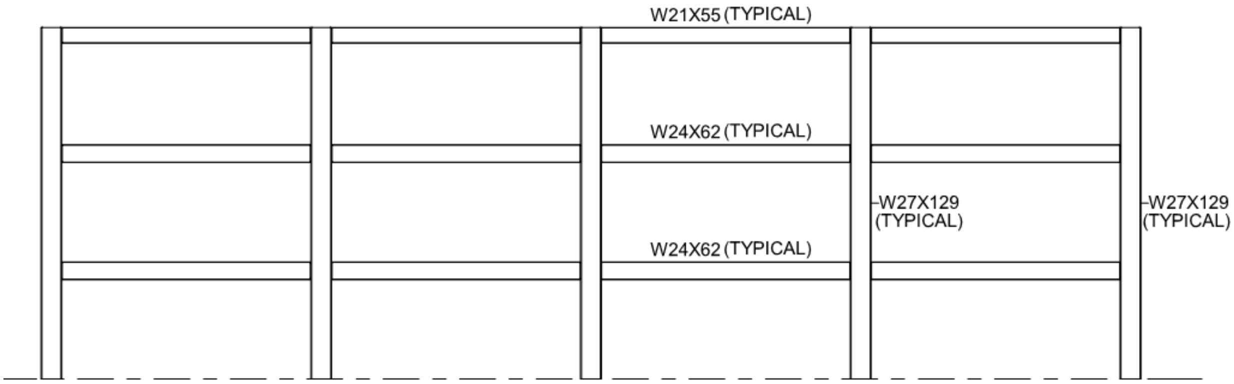


Figure 22 - Moment Frame Elevation

For constructability, the moment frame system, or control system, had strengths. . The erection and construction of moment frames are familiar and the infield requirements were limited to bolting which improves constructability. However, either individual pieces of steel are quite large. Each column in the system was approximately 40 ft. tall and weighed 2.5 tons. For the masonry, the infill panel was relatively generic. Tolerances for infill, reinforcement, and connections posed the biggest concerns, but those tolerances are typically present in any masonry system

The moment frame, or control system, had fair redundancy in this project. Since multiple bays of moment frames in a row were controlled by deflection, if one item “failed” and was removed for this exercise, the entire system did not encounter instability. The items that could fail in this system were individual components of connections, beams, girders, and columns. The connection was designed to withstand the maximum moment possible in the system. This design methodology and resistance factors instilled a reserve capacity in the system in which if one item in the connections failed, it was still capable of resisting the required loads. When one beam was removed, the other members of the system had enough capacity to resist the lateral load, although the drift increased approximately 0.25 in. at the second floor, or the governing story. The only lateral case that caused global instability for the frame was when a column was removed. However, if the moment frame system was comprised of a single bay, the system would only have redundancy in the individual components of each connection. For redundancy in regards to gravity, the system did not perform as well. With only infill-type masonry, a failing beam does not have anything to guide it to the top of the wall. Even if it rests on the top of the infill panel, the top of the masonry is not required to be a load distributing member. Without load distribution the probability of single units dislodging increases.

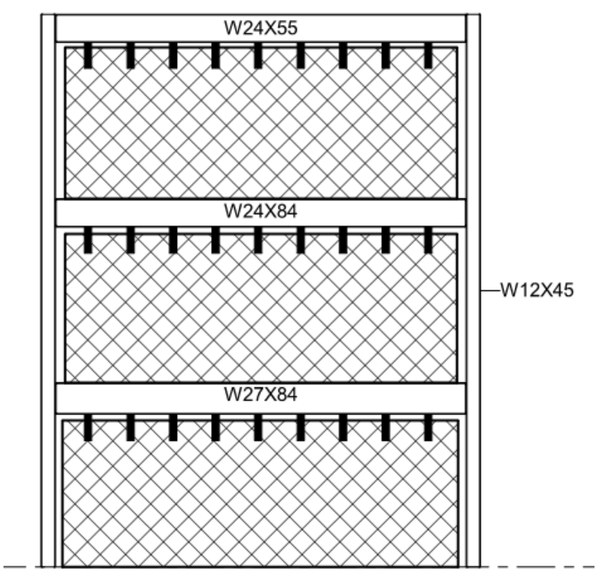
Materials for the moment frame lateral, or control, system were minimal, with the exception of the required quantity of structural steel and welding. For the masonry, the minimum quantity of units, reinforcement, and grouting were used due to the infill nature of the panel. This minimum quantity amounted to approximately 18,250 individual units, 4,330 lbs of reinforcement, and 38 cu yd of grout. The foundation for the system was based on the IBC minimum allowable soil-bearing capacity of 1,500 psf, requiring sizable footings of approximately 500 cu yd of concrete with 32.5 tons of reinforcement. The structural steel required for the lateral system weighed approximately 111 tons, including connections, which is approximately 1.5 times more than the same quantity of similar gravity members. The moment frame design also required approximately 2,600 equivalent feet of welding, most of which was full-penetration welds. Fortunately, all welding could of been done in the steel fabrication shop, thereby increasing the welding quality and decreasing welding costs. Field welding is expensive because of the work conditions and inspections required, so limiting those welds significantly helps the economics of the system. Appendix D contains more in-depth material usage explanations.

For the comparison matrix, the moment frame system was used as the standard, so all attribute values were equal to zero or no change. The last section of this chapter contains a graphical comparison of all three systems.

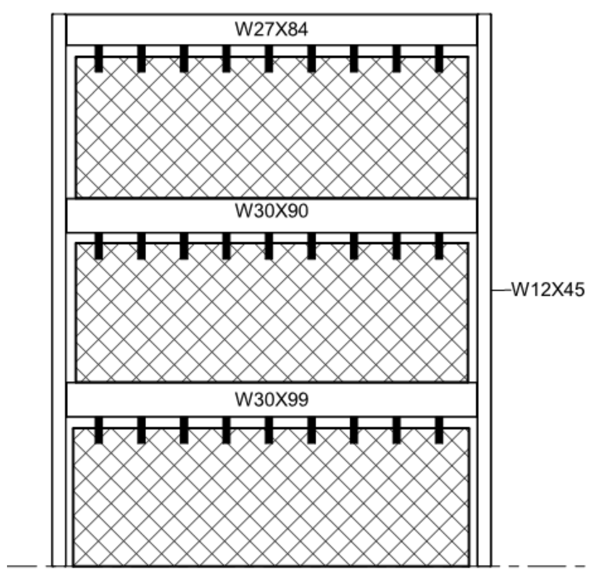
Type I Hybrid Masonry Design

Design of the Type I hybrid masonry system includes structural steel, masonry panels, and steel plate links. The layout was globally dictated by links between the steel and masonry. Two bays of hybrid Type I masonry were required on each side of the building in order to resist seismic forces. The required amount of links at the first level was greater than could fit on one panel. For the individual members, beams and girders in the system were governed by heavy loads and tight deflection limits imposed by the masonry. At the second story, deflection increased due to additional grout in the masonry panel to resist overturning. The typical bay for a beam or secondary member condition included W12x45 columns with W24x55, W24x84, and W27x84 beams from the roof down as shown in Figure 23(a). The girder condition bays required

W27x84, W30x90, and W30x99 girders from the roof down as shown in Figure 23(b). The beams were connected to the columns with simple 0.375 in. shear tabs with six .75 in diameter bolts each. Unlike masonry in the moment frame system, the panels had to resist significant amounts of in-plane shear, thereby requiring additional detailing for the hybrid Type I panels. Hybrid masonry panels on the third floor resisted the least lateral load and were grouted and reinforced at the minimum level with N0. 5 rebar at 48-in. spacing. Panels on the first and second floors required full grouting to develop the required overturning resistance. Finally, links and welded reinforcement, items unique to this system, were included. A total of approximately 240 pairs of links were required. The third floor required six links per panel, and the first floor of the structure required 29 links.



(a) Beam Condition



(b) Girder Condition

Figure 23 - Type I Hybrid Masonry Frame Elevations

For constructability of the Type I hybrid masonry system, additional links were the biggest concern. The link required attachment to both the beam and the masonry panel. The beam connection was field-welded or bolted to the flange; bolting was preferred to ease construction and limit field welding. For each link the panel also had to be drilled through to attach the links with a through bolt. Drilling through 8 in. of concrete is time-consuming and difficult but common.

Construction of the masonry panel was significantly more complex. Because the reinforcement had to transfer tension to the beam, it must be lapped or coupled. A bond beam was also required at the top of the masonry panel in tight construction space (Moreels, 2016). In addition, all previously mentioned hindrances of typical masonry infill were still factors. Therefore, masonry constructability was given a significant disadvantage rating, as reflected in the comparison matrix at the end of this section.

Steel constructability for Type I hybrid masonry was comparable to the standard moment frame system. Although the heaviest member was significantly smaller than the heaviest member of the moment frame system, the average members were heavier. The heaviest member for the Type I hybrid masonry system was a 30-ft. long girder weighing slightly less than 1.5 tons. This girder only had to be raised 13 ft. above finished floor, but typical beams and girders of the hybrid system are approximately 35 and 65 percent heavier than comparable moment frame pieces, respectively. Moment frame beams were smaller than the simply supported beams in the Type I hybrid system because the fixed connections reduce deflection, which is the governing factor for Type I hybrid masonry beams.

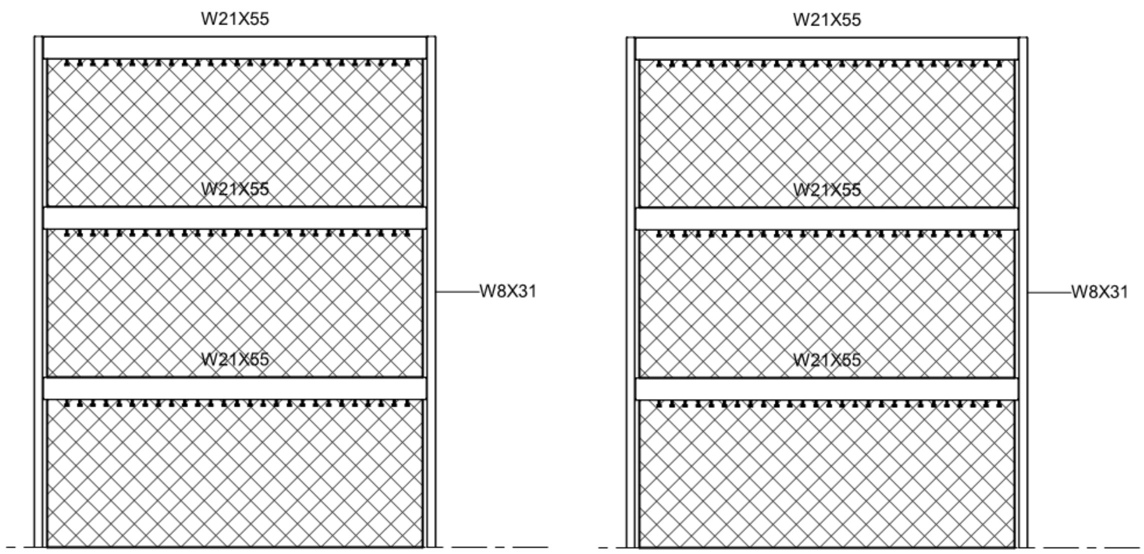
The redundancy score for Type I hybrid masonry suffered because of the individual bay layout for this design. Individual bays allowed openings at the center of each face of the building while maintaining a symmetrical layout. Contributing items to this system included beams, columns, masonry panels, shear tab connections, and masonry links. The most likely item to fail prematurely in the lateral system by quantity was a link. Since the number of links was based on preventing yielding with a safety factor of approximately 1.5, if one link failed, the other links resisted the displaced load. The system faired very well for redundancy in the system regarding gravity. Unlike the moment frame infill masonry, Type I hybrid masonry contains a bond beam at the top of the panel to provide better anchorage and load distribution for the links as well as distribute gravity loads from a beam. The connection of the beam to the wall via the links increases the likelihood of a failing beam directly bearing on the hybrid masonry panel. Beams in the Type I hybrid masonry system are also governed by strict deflection limits, so they have considerable reserve capacity. The connection for the beam to column also has reserve capacity because the governing factor for its design was the width of the beam requiring contact with the shear tab. Overall, the Type I hybrid masonry system performed significantly better than the moment frame system.

Material requirements for the Type I hybrid masonry system ranged from similar to significantly worse than the moment frame system, with the exception of required overall structural steel and welding. For the masonry, approximately the same quantity of units and reinforcement were required as in the moment frame system, but significantly more grout was required for the Type I hybrid masonry system on the order of nearly 1.5 times more grout by volume, amounting to approximately 18,804 individual units, 4,724 pounds of reinforcement, and 85 cu yd of grout. Foundations for the system were based on the same allowable soil-bearing capacity of 1,500 psf as the control system, resulting in larger footings than the moment frame system which require approximately 608 cu yd of concrete with 42 tons of reinforcement. The total structural steel required for the lateral system was less than the total structural steel required for the moment frame system primarily due to heavy structural connections of the moment frame. Structural steel for Type I hybrid masonry systems weighed approximately 78 tons, including connections that were approximately 30 percent less than the moment frame system. For connections, the Type I hybrid masonry system fell on both ends of the spectrum. The system required approximately 16 percent more bolts than the moment frame system, but the required welding was significantly less. Shop welding required for this system was approximately 325 ft. of welding, which was an 89 percent improvement from the moment frame system. However, Type I hybrid masonry panels required the reinforcement to be field-welded to the supporting beam, adding approximately 424 in. of field-welding to the project. More in-depth material usage explanations are included in Appendix D.

The final section of this chapter contains an overall comparison of all three systems in a quantitative comparison matrix.

Type II Hybrid Masonry Design

Design of the Type II hybrid masonry system included structural steel, masonry panel, and shear studs. The layout was dictated by the required number of shear studs between the steel and masonry. Two bays of Type II hybrid masonry were required on each side of the building in order to resist seismic forces. The required amount of studs at the first level was greater than could fit on one panel. For individual members, beams and girders in the system were governed by the flexural strength of the beams supporting floor dead and construction live loads. The beams did not have to meet the strict deflection criteria of previous systems because they did not support the weight of the masonry. Self-supported masonry allowed the beams to be considerably smaller than the moment frames or Type I hybrid masonry. The typical bay for this system included W8x31 columns with W16x31 beams and W21x55 girders as shown in Figure 24.



(a) Beam Condition (b) Girder Condition

Figure 24 -Type II Hybrid Masonry Frame Elevations

The beams were connected to the columns with 0.375 in. shear tabs with four .75 in diameter bolts. Unlike masonry in the moment frame system, the panels had to resist significant amounts of in-plane shear, and unlike Type I hybrid masonry, these panels supported significant dead and live loads. As the overturning moment in the panels increased closer to the base of the building, the axial dead load also increased. The additional dead load provided significant resistance to overturning and therefore reduced grouting requirements for the panels as compared to Type I panels. Type II hybrid masonry panels resisted required lateral load with minimum reinforcing, which was No. 5 rebar and grouted cells with 48-in. spacing. The unique item to this system was the shear studs. Studs with diameters of 1 in. were welded to the top and bottom flanges of the beam to transfer shear between the beam and masonry panel. The number of studs depended on the shear at the corresponding level. Twenty-nine studs were required on the bottom flange of the roof beams. The third-floor beam required 29 studs on the top flange and 52 studs on the bottom flange. The second-floor beam required 52 studs on the top flange and 63 studs on the bottom flange

For constructability of the Type II hybrid masonry system, the sequencing of construction is the biggest concern. Each floor's masonry panel may only be constructed after all panels in the floors below has sufficient strength to support the new panel. Only approximately four courses of masonry block are typically constructed at a time. The masonry requires hours to set before an additional set of courses can be laid without squashing the joints freshly laid joints (Moreels, 2016). In both of the other systems in this study the masonry at each floor can be laid simultaneously because the masonry panels do not rely on each other for vertical support. Waiting to chronologically construct all the masonry compromises a project's schedule. Type II hybrid masonry panels also require shear studs to be welded between the interfaces of the beam and masonry panel. Although welding is relatively easily performed with welding guns, placement of these studs in regards to the bottom layer of masonry requires considerable coordination. Limiting stud spacing to modular dimensions or providing additional sacrificial studs that may be removed could improve the constructability. The top of the masonry panel also requires special treatment. Formwork was required to create an additional solid course to the masonry where the studs on the bottom flange of the beam directly above the panel embed. The top and bottom course of the hybrid panel must be fully grouted full, but the rest of the masonry panel requires no more complex construction than the typical masonry infill panel. Therefore, masonry constructability was given a minor disadvantage rating, as reflected in the comparison matrix at the end of this section.

In summary, the steel constructability for this system was immensely better than the standard moment frame system. Girders were either the same weight or 10 percent less than the similar moment frame member, depending on the floor. Beams were 50 percent lighter than the moment frame counterpart. The largest difference between the Type II hybrid steel components and the moment frames was the column weights. The Type II hybrid masonry column provided a 76 percent decrease in column weight. The heaviest member for the Type II hybrid masonry system was a 30-ft. long girder weighing less than 1 ton, as compared to 2.5 tons for the moment frame system. The significantly smaller steel weights could allow for a smaller, less expensive crane on the job site depending on many side specific details.

The redundancy score for Type II hybrid masonry suffered due to the individual bay layout for this design. Individual bays allowed openings at the center of each face of the building while maintaining a symmetrical layout. Contributing items to this system included beams, columns, masonry panels, shear tab connections, and shear studs. According to quantity, the most likely item to fail was a shear stud since there are many more of them than the other components. Because design of the studs neglected friction between the masonry and steel beam, additional capacity should be available to account for a defective stud.

For redundancy in the system in regards to gravity, the system performed better than the other two systems in the study. The masonry supported gravity loads originally, thereby creating advantages and disadvantages for redundancy. If a beam or shear tab connection fails, the dead load of the floor automatically redistributed to the masonry as live loads already follow this load path. When loads track into the masonry panels, less reserve axial capacity is available; however, this should not be a concern in typical low-rise buildings because the masonry walls were not not governed by compression. The masonry panel in this study used approximately 10 percent of the available capacity. Overall, the Type II hybrid masonry system performed significantly better than the moment frame system.

Material and economic requirements for the Type II hybrid masonry system ranged from similar to impressively better than the moment frame system, with the exception of time requirements of the masonry. Relatively the same quantities of units and reinforcement are required for the masonry compared to the previous two systems. A slight increase of less than 10 percent volume of grout is required for the Type II hybrid masonry system compared to the moment frame system. The extra grout provided solid top and bottom courses of each panel. Total masonry panel requirements amounted to approximately 19,150 individual units, 4,900 pounds of reinforcement, and 40 cu yd of grout. The foundation for the system was based on the same allowable soil-bearing capacity as the previous two systems of 1,500 psf, resulting in footings somewhat smaller than the moment frame system. The foundation system required approximately 400 cu yd of concrete with 26 tons of reinforcement. Total structural steel required for the Type II hybrid system was impressively less than the total structural steel required for the moment frame system. Structural steel for Type II hybrid masonry systems weighed approximately 58 tons, including connections that demonstrated an approximate 50 percent reduction in steel.

For structural steel connections, the Type II hybrid masonry system required approximately 1,000 or 70 percent less bolts, 90 percent less welds by equivalent length, and 98 percent less additional plates by weight than the moment frame system. However, Type II hybrid masonry panels required approximately 1,800 studs to be welded in the field, adding approximately 424 in. of field welding to the project. Appendix D contains in-depth material usage explanations and comparisons. Depending on details of individual projects and the organization and practices of the contractor, all discussed material advantages could be outweighed by time required for the masonry.

The final section of this chapter contains an overall comparison between all three systems in a quantitative comparison matrix.

Systems Comparison

The moment frame, Type I hybrid masonry, and Type II hybrid masonry lateral systems have unique advantages and disadvantages. Some of these advantages are relatively easy to compare while others are more complicated or based on judgement calls and personal preference. This can make comparing different options difficult. to compare. First the systems were quantified by material usage Table 7 summarized the materials required for each of the three systems. More detailed material requirements for the three systems are available in Appendix E.

Table 7- Material Usage Comparison

	Structural Steel		Masonry		Concrete				Reinforcement		Connections						
											Bolts		Welds (shop)		Welds (field)	Studs	Links
Lateral System	tons	Improvement	Units	Improvement	Concrete (cy)	Improvement	Grout (cy)	Improvement	tons	Improvement	Quantity	Improvement	Length (in)	improvement	Length (in)	Quantity	Quantity
Moment Frame (standard)	111	0%	18240	0%	505	0%	38	0%	35	0%	1536	0%	31236	0%	0	0	0
											0						
Hybrid Masonry Type I	78	30%	18804	-3%	608	20%	86	-127%	44	-28%	1776	-16%	3456	89%	424	0	240
											0						
Hybrid Masonry Type II	58	48%	19148	-5%	392	22%	41	-7%	28	19%	480	69%	2880	91%	0	1800	0

The following matrix quantifies attributes of the three systems. The moment frame system is portrayed the standard practice by which the two hybrid systems are judged. Positive values in the matrix indicate an improvement and negative values indicate a setback. The magnitude of the score is the severity of the change. The magnitudes of the values are limited to one for minor changes, two for significant changes, and three for outstanding changes. Although this system is somewhat subjective, it provided an adequate overall comparison of all three systems. The information that the scores are based on is all present within this report and appendixes for reference. Just as one system is not the best solution for every project, not every designer has the same priorities. The matrix can be adjusted by applying importance factors to items that the designer feels are more important than others.

Three main proposed advantages affect the overall performance of the three lateral systems: constructability, design, and economics. Each category can be subdivided into multiple subcategories. Care was taken to keep the possible number of points in each category relative to the importance of that category and minimize overlap.

The first category of constructability was divided into the following five subcategories:

1. Maneuverability described the size, weight, and ease of moving construction materials, especially steel, for the system.
2. Steel connection ease described the degree of time and skilled labor to install connections between structural steel members in the system.
3. Masonry ease described the time and skilled labor to lay masonry panels for the system.
4. Masonry connection ease described the degree of time and skilled labor required to install connections between masonry panels and structural steel.
5. Familiarity described the amount of additional training workers may require to properly construct a system.

The second category of design was subdivided into the following three categories:

1. Ease of design described the time and skill involved to design the system.
2. Lateral redundancy described the system's ability to cope with minor failures to the lateral system.
3. Gravity redundancy described the system's ability to cope with minor failures related to vertical dead and live loads.

The final category of economics was subdivided into the following five categories:

1. Structural steel described the required quantity of steel for the system by weight.
2. Masonry described the required quantity of material for masonry panels of the system; the score includes masonry units and grout.

3. Foundation described the volume of concrete required to support the lateral system.
4. Connection material described the quantity of individual parts for the construction of connections for the system, including the weight of structural steel connections and links as well as bolt and stud quantities.
5. Connection time describes laborious and skilled requirements of some connections, primarily welding or items with high coordination concerns.

The completed comparison matrix, shown in Table 8, allows comparison of subcategories and main categories by presenting the individual or sum total of the values. Considering all categories and subcategories, the data suggests that differences between a moment frame system with masonry infill and a Type I hybrid masonry system are negligible. The optimal solution for the project must be determined by personal preferences and priorities of the design and constructions team. Type II hybrid masonry, however, has a considerable positive value, indicating that the system should at least be considered. As mentioned, project scheduling and time requirements significantly impact Type II hybrid system's applicability. A fast-paced or poorly organized project would likely not benefit from this system.

Table 8- Comparison Matrix

	Constructability					Design			Economics					
	Manuverability	Steel connection ease	Masonry panel ease	Masonry connection ease	Familiarity	Ease of design	Lateral redundancy	Gravity redundancy	Structural steel	Masonry	Foundation	Connection materials	Connection time	**Result**
Moment Frame (standard)	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hybrid Masonry Type I	-1	3	-2	-2	1	-1	1	1	1	-2	-1	1	1	**0**
Hybrid Masonry Type II	3	3	-3	-2	-1	-2	1	1	3	-3	1	2	2	**5**

Chapter 5 - Conclusion

Constant new developments in the construction industry require that the emerging systems' applicability be evaluated;. positive innovations should be implemented and negative innovations tabled for further improvements. This is precisely what this study aimed to accomplish by investigating the use of hybrid masonry in seismic lateral force resisting systems for low-rise buildings. Hybrid masonry was compared to the standard practice of using moment frames around required masonry infill to resist lateral forces. Origins of the hybrid system were initially examined, followed by explanation of design practices, and presentation of a sample study. The sample study was designed using identical buildings with three different lateral systems: an ordinary moment frame with masonry infill system, a Type I hybrid masonry system, and a Type II hybrid masonry system. thereby allowing system comparisons based on attributes of constructability, design, and economics.

Results of the comparison between the moment frame with masonry infill, Type I hybrid masonry, and Type II hybrid masonry emphasized the fact that there is always more than one solution to a problem, although some solutions may be better suited to a particular problem than others. In this study, Type II hybrid masonry showed promise as a solution to the steel building with a cavity wall scenario. Significant reductions in steel member sizes and the complexity of connections contributed to the success of this system in the study. Another key to the success was the system's ability to utilize the strength of masonry in compression. However, the primary drawback to this system is time because masonry panels in the hybrid masonry system must be built from the ground up, whereas the other systems allow simultaneous construction on all floors. This requirement also drastically affects a project's construction schedule. The two remaining systems were similar in overall evaluations, but they demonstrated unique strengths and weaknesses. The moment frame is traditional and well understood, but it has expensive, time-consuming fixed connections. Type I hybrid masonry eliminates these connections but requires more material and complications for masonry construction. Each system could be the optimal solution to individual situations.

As with any study, this program had limitations and requires investigation of additional items. A missing component to this study was converting the study to one overall unit, that would help scale the results to the market’s current priorities, but it is outside the scope of this parametric study.

Chapter 6 - Bibliography

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Appendix A - Seismic Load Determination

Hybrid Masonry

SEISMIC LOAD- Kansas City, MO (Hybrid Masonry - Ordinary Masonry Controlled)				REFRENCES
Site class	D	(assumed)		ASCE 7-10 T20.3-1
Category	II			
Importance factor	1			ASCE 7-10 T1.5-2
Site Locations:	Kansas City, MO			
Spectral Accelerations				
S_s=	0.112 g			http://geohazards.usgs.gov/designmaps/us/application.php
S_1=	0.064 g			
Site Coefficients				
F_a=	1.6			ASCE 7-10 T11.4-1
F_v=	2.4			ASCE 7-10 T11.4-2
S_{MS}=	0.179			ASCE 7-10 11.4-1
S_{M1}=	0.154			ASCE 7-10 11.4-2
S_{DS}=	0.119			ASCE 7-10 11.4-3
S_{D1}=	0.103			ASCE 7-10 11.4-4
Seismic Design Catergory				
Short period	A			ASCE 7-10 T11.6-1
1-S period	B	(governs)		ASCE 7-10 T11.6-2

EQIVALENT LATERAL FORCE PROCEDURE

EFFECTIVE SEISMIC WEIGHT — ASCE 7-10 12.7.2

Roof:	Dead Load	82 psf		(dead load calcs)
	Snow	0 psf	($P_f < 30$ psf)	exception 4
		82 psf		
	Area	21600 ft^2		
	DL_{roof}=	**1771.20 kips**		
Floor:	Dead Load	78 psf		(dead load calcs)
	Partition	10 psf		exception 2
		88 psf		
	Area	21600 ft^2		
	DL_{floor}=	**1900.80 kips**		

Wall:	Dead Load	83 psf		
	Perimeter	600 ft		
		Trib. Width	DL_w	
	roof	6.5 ft	**323.70 kips**	$=DL_{WR}$
	standard	13 ft	**647.40 kips**	$=DL_{Ws}$
	1st elev.	13 ft	**647.40 kips**	$=DL_{W1}$
	ground	6.5 ft	**323.70 kips**	$=DL_{W0}$
	elevated floors including roof=	3		

Total W= 7515 kips

SHEAR WALLS

R=	2	(Ordinary Reinforced masonry shear wall)	ASCE 7-10 T12.2-1
fundamental period			
C_t=	0.02	(all other structural systems)	ASCE 7-10 T12.8-2
x=	0.75	(all other structural systems)	ASCE 7-10 T12.8-2
h_n=	39	ft	ASCE 7-10 11.2
T_a=	0.312	s	ASCE 7-10 12.8-7
long-period			
T_L=	12	s T< TL USE 12.8-3	
C_s need not exceed=		0.165	ASCE 7-10 12.8-3
$C_{s\ calc}$=	0.060		ASCE 7-10 12.8-2
C_s shall not be less than=		0.01	ASCE 7-10 12.8-5
C_s shall not be less than=		0.016 (S_1>/=.6g)	ASCE 7-10 12.8-6
USE C_s=	0.060		
BASE SHEAR= C_sW=	**447.1 kips**		ASCE 7-10 12.8-1
k=	1	(period < .5 s)	ASCE 7-10 12.8.3

Level	h_x (ft)	w_x (kips)	$w_xh_x^k$	C_{vx}	F_x (/level)	V_x (kips)	ASCE 7-10 12.8-11
ground							ASCE 7-10 12.8-12
1st elev.	13	2548	33127	0.1829	81.80	**447.14**	
2nd elev.	26	2548	66253	0.3659	163.60	**365.34**	
roof	39	2095	81701	0.4512	201.74	201.74	
		sum=	**181081**	CHECK			

Accidental Torsion

.05L= 9.000 ft

Level	Frames	M_t (k-ft)	Couple (kips)	per panel (kips) Fx	Vx	links req'd
ground	4					
2nd Floor	4	736	6.1	23.5	128.6	13.00
3rd Floor	4	1472	12.3	47.0	105.0	11.00
roof	4	1816	15.1	58.0	58.0	6.00

Ordinary Steel Moment Frame

SEISMIC LOAD- Kansas City, MO (ordinary steel momet frame)			REFRENCES
Site class	D	(assumed)	ASCE 7-10 T20.3-1
Category	II		
Importance factor	1		ASCE 7-10 T1.5-2
Site Location:	Kansas City, MO		
Spectral Accelerations			
S_s=	0.112 g		http://geohazards.usgs.gov/designmaps/us/application.php
S_1=	0.064 g		
Site Coefficients			
F_a=	1.6		ASCE 7-10 T11.4-1
F_v=	2.4		ASCE 7-10 T11.4-2
S_{MS}=	0.179		ASCE 7-10 11.4-1
S_{M1}=	0.154		ASCE 7-10 11.4-2
S_{DS}=	0.119		ASCE 7-10 11.4-3
S_{D1}=	0.103		ASCE 7-10 11.4-4
Seismic Design Catergory			
Short period	A		ASCE 7-10 T11.6-1
1-S period	B	(governs)	ASCE 7-10 T11.6-2

EQIVALENT LATERAL FORCE PROCEDURE

EFFECTIVE SEISMIC WEIGHT				ASCE 7-10 12.7.2
Roof:	Dead Load	82 psf		(dead load calcs)
	Snow	0 psf	(P_f < 30 psf)	exception 4
		82 psf		
	Area	21600 ft^2		
	DL_{roof}=	**1771.20 kips**		
Floor:	Dead Load	78 psf		(dead load calcs)
	Partition	10 psf		exception 2
		88 psf		
	Area	21600 ft^2		
	DL_{floor}=	**1900.80 kips**		

Wall:	Dead Load	83 psf		
	Perimeter	600 ft		
		Trib. Width	DL_w	
	roof	6.5 ft	**323.70 kips**	$=DL_{WR}$
	standard	13 ft	**647.40 kips**	$=DL_{Ws}$
	1st elev.	13 ft	**647.40 kips**	$=DL_{W1}$

elevated floors including roof= 3

Total W= 7191 kips

SHEAR WALLS

R=	3.5	(Ordinary Steel Moment Frame)	ASCE 7-10 T12.2-1

fundamental period

C_t=	0.028	(steel moment frame)	ASCE 7-10 T12.8-2
x=	0.8	(steel momemt frame)	ASCE 7-10 T12.8-2
h_n=	39	ft	ASCE 7-10 11.2
T_a=	0.525	s	ASCE 7-10 12.8-7

long-period

T_L= 12 s T< TL USE 12.8-3

C_s need not exceed=	0.056	ASCE 7-10 12.8-3
$C_{s\ calc}$=	0.034	ASCE 7-10 12.8-2
C_s shall not be less than=	0.01	ASCE 7-10 12.8-5
C_s shall not be less than=	0.009143 (S_1>/=.6g)	ASCE 7-10 12.8-6

USE C_s= 0.034

BASE SHEAR= C_sW= 244.5 kips ASCE 7-10 12.8-1

k= 1 (period < .5 s) ASCE 7-10 12.8.3

Level	h_x (ft)	w_x (kips)	$w_x h_x^k$	C_{vx}	F_x (/level)	V_x (kips)	ASCE 7-10 12.8-11
ground							ASCE 7-10 12.8-12
1st elev.	13	2548	33127	0.1829	44.73	**244.50**	
2nd elev.	26	2548	66253	0.3659	89.46	**199.78**	
roof	39	2095	81701	0.4512	110.32	110.32	
		sum=	**181081**	CHECK			

Accidental Torsion

.05L= 9.000 ft

Level	Frames	M_t (k-ft)	Couple (kips)	per frame (kips) Fx	Vx
ground					
1st elev.	2	403	3.4	25.7	135.4
2nd elev.	2	179	1.5	46.2	109.7
roof	2	993	8.3	63.4	63.4

Appendix B - Wind Calculations

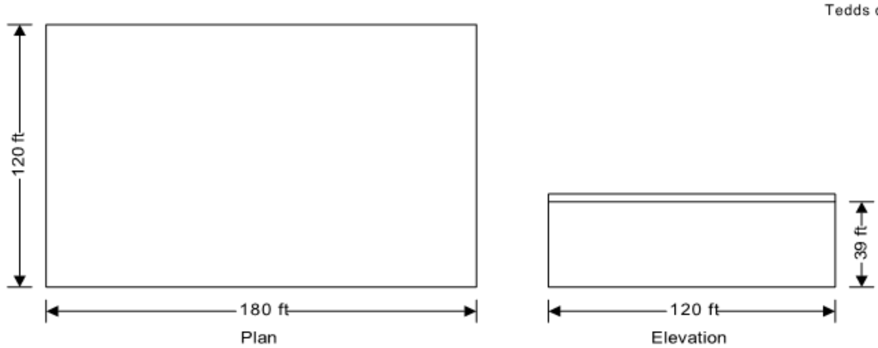
Main Wind Force Resisting System Load Determination

WIND LOADING (ASCE7-10)

In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method

Tedds calculation version 2.0.15



Building data

Type of roof	Flat
Length of building	b = **180.00** ft
Width of building	d = **120.00** ft
Height to eaves	H = **39.00** ft
Height of parapet	h_p = **3.50** ft
Mean height	h = **39.00** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coeff (Table 26.6-1)	K_d = **0.85**
Exposure category (cl.26.7.3)	B
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.57	16.40
25.00	0.66	18.99
39.00	0.75	21.70
42.50	0.77	22.23

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **21.70** psf

Parapet pressures and forces

Velocity pressure at top of parapet	q_p = **22.23** psf
Combined net pressure coefficient, leeward	GC_{pnl} = **-1.0**
Combined net parapet pressure, leeward	$p_{pl} = q_p \times GC_{pnl}$ = **-22.23** psf

Combined net pressure coefficient, windward — GC_{pnw} = **1.5**

Combined net parapet pressure, windward — $p_{pw} = q_p \times GC_{pnw}$ = **33.35** psf

Wind direction 0 deg:

Leeward parapet force — $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-14** kips

Windward parapet force — $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **21** kips

Wind direction 90 deg:

Leeward parapet force — $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-9.3** kips

Windward parapet force — $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **14** kips

Pressures and forces

Net pressure — $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force — $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	39.00	-0.90	21.70	-20.50	3510.00	-71.97
B (-ve)	39.00	-0.90	21.70	-20.50	3510.00	-71.97
C (-ve)	39.00	-0.50	21.70	-13.13	7020.00	-92.15
D (-ve)	39.00	-0.30	21.70	-9.44	7560.00	-71.36

Total vertical net force — $F_{w,v}$ = **-307.46** kips

Total horizontal net force — $F_{w,h}$ = **0.00** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.40	7.25	2700.00	19.57
A_2	25.00	0.80	18.99	9.01	1800.00	16.22
A_3	39.00	0.80	21.70	10.85	2520.00	27.34
B	39.00	-0.50	21.70	-13.13	7020.00	-92.15
C	39.00	-0.70	21.70	-16.82	4680.00	-78.70
D	39.00	-0.70	21.70	-16.82	4680.00	-78.70

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_0} = b \times (H + h_p)$ = **7650.00** ft^2

Projected vertical area of roof — $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **122.4** kips

Leeward net force — $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-106.2** kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wpw\_0}$ = **84.1** kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **190.3** kips

The wind base shear of 190.3 kips is less than the seismic base shear of 447.1 kips for hybrid masonry and 224.k kips for ordinary steel moment frames therefore seismic governs for all cases in the parametric study.

Masonry Out of Plane Flexural Design

Wall Properties

Location=			
Material=	conc		
Mortar type=	S		
grouted fr=	163	psi	(TMS 402-11 T3.1.8.2)
ungrouted fr=	63	psi	(TMS 402-11 T3.1.8.2)
interpulated fr=	79.7	psi	
Nominal thickness, t=	8	in	
True thickness, t_{sp}=	7.625	in	
Wall height, h=	11	ft	
f'm=	1500	psi	
E'm=	1350000	psi	(TMS 402-11 1.8.2.2.1)
Eccentricity, e=	0	in	by min. bearing length (joist)
Try:	#5 @ 48" SPA.		
A_n=	40.7	in^2/ft	(NCMA TEK 14-1A)
I_n=	332	in^4/ft	(NCMA TEK 14-1A)
S_n=	87.1	in^3/ft	(NCMA TEK 14-1A)
r=	2.66	in	(NCMA TEK 14-1A)
Slenderness, h/t=	16.5		
Slenderness, h/r=	49.6		

Loading

Verticle Loads

Wall weight=	83	psf	
Dead Load, P_{DW}=	456.5	lb/ft	@ mid height
Dead Load, P_{Df}=	0	lb/ft	
Live Load, P_L=	0	lb/ft	
Roof Live Load, P_{LR}=	0	lb/ft	
Snow Load, P_S=	0	lb/ft	
Rain Load, P_R=	0	lb/ft	
D=	456.5	lb/ft	
D+L=	456.5	lb/ft	
D+(Lr or S or R)=	456.5	lb/ft	
D+.75L+.75(Lr or S or R)=	456.5	lb/ft	
governing case, Pa=	**456.5**	**lb/ft**	
1.4D=	639.1	lb/ft	
1.2D+1.6L+.5max(Lr,S,R)=	547.8	lb/ft	
1.2D+1.6max(Lr,S,R)+L=	547.8	lb/ft	
1.2D+1.0W+L+.5max(Lr,S,R)=	547.8	lb/ft	
1.2D+1.0E+L+.2S=	547.8	lb/ft	

.9D+1.0W=	410.85	lb/ft	
.9D+1.0E=	410.85	lb/ft	
governing case, Pu=	**639.1**	**lb/ft**	@ mid height

Lateral Loads

Wind Load, W=	22.3	psf	(ASCE 7-10)
D=	0	lb/ft	
D+L=	0	lb/ft	
D+(Lr or S or R)=	0	lb/ft	
D+.75L+.75(Lr or S or R)=	0	lb/ft	
D+max(.6W,.7E)=	13.38	lb/ft	
D+.75L+.75(0.6W)+0.75(Lr,S,R)=	10.035	lb/ft	
D+.75L+.75(0.7E)+0.75S=	0	lb/ft	
.6D+.6W=	13.38	lb/ft	
.6D+.7E=	0	lb/ft	
governing case, wa=	**13.38**	**lb/ft**	

Reinforcement

Bar=	#5	
d_b=	0.625	in
A_b=	0.31	in^2
fy=	60	ksi
Spacing=	48	in o.c.
Area/ft=	0.0775	in^2
d=	3.8125	in
n=	21.48148	

Axial Capacity-@ base

ϕ=	0.9		(TMS 402-11 3.1.4.4)
Slenderness factor=	0.87436		(TMS 402-11 3.3.4.1.1)
ϕP_n=	33615.5	lb	

ϕPn>Pu, OK

Pu/Ag<.2f'm	yes, valid
h/t < 30	yes, neglect slenderness

Moment Capacity

$M_a=w_u h^2/8$=	4047	lb-in	
$M_e=P_{uf}(e/2)$=	0	lb-in	
M_{u1}=	4047	lb-in	
δ_u=	0.00	in	
$M_\delta=P_u\delta$=	0	in-lb	
I_{cr}=	21.68	in^4	(TMS 402-11 3-25)
c=	0.459	in	OK, in face shell
M_{cr}=	6939	lb-in	

$a=$	0.372	in
$M_{u2}=$	4047	in-lb
$\phi=$	0.90	
$M_n=$	19438	in-lb
$\phi M_n=$	17494	in-lb

ϕMn>Mu, OK

Deflection Check

$\delta_{limit}=$	0.924	in	(TMS 402-11 3.3.5.5)
$M_{serv}=$	2428.47	lb-in	
$\delta_s=$	0.0281	in	

OK

Appendix C - Sample Design for a Type I Hybrid Masonry Frame

Hybrid Type I - Girder Case - Roof Girder

Infill Spacing=	10 ft		
Infill Span=	30 ft		
Girder Span=	30 ft		
Wall dead load=	83 psf		
Wall Height=	4 ft		
Wall dead load=	--- ---	332	plf
Floor Dead Load=	78 psf	0	k/beam
Roof Dead Load=	82 psf	12.3	k/beam
Live Load=	0 psf	0	k/beam
Roof Live Load=	20 psf	3	k/beam
Total Seismic Load=	58 kips (horizontal)		
Design Spectral Response Acceleration, SDS=	0.119		
Redundancy factor=	1		

M=(wl2/8+Pa)*(appropriate factors)=

V=(wl/2+P)*(appropriate factors)=

Beam Loads		Moment	Shear
1)	1.4D=	224.49 k-ft	24.2 kips
2)	1.2D + 1.6L + .5Lr=	207.42 k-ft	22.2 kips
3)	1.2D + 1.6Lr + .5L=	240.42 k-ft	25.5 kips
4)	$(1.2 + .2S_{DS})D + \rho Q_E + Lr=$	196.24 k-ft	21.1 kips
5)	$(.9 - .2\ S_{DS})D + \rho Q_E=$	140.50 k-ft	15.1 kips

Axial Loads		
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L=$	29.00 kips (max)
5)	$(.9 - .2\ S_{DS})D + \rho Q_E=$	29.00 kips (max)

Calculated (refrence shear, moment, and axial diagrams)

Largest Flexural Load:	@ mid span
V_u=	0 kips
P_U=	14.5 kips
M_U=	240 k-ft
Largest Axial Load:	@ start of span
V_u=	21 kips
P_U=	29.00 kips
M_U=	0 k-ft

Try	27x84	AISC T 1-1
E=	29000 ksi	
F_y=	50 ksi	

Compressive Strength

K_x=	1 (pin-pin)	AISC TC-A-7.1
L_x=	1 ft	
slenderness=	1.12	
K_y=	1 (pin-pin)	AISC TC-A-7.1
L_y=	10 ft	
slenderness=	57.97 ***governs***	

$4.71*SQRT(E/F_y)$=	113.43		USE E3-2	
F_e=	85.17	ksi	AISC 360 E3-4	
F_{cr}=	39.11	ksi	AISC 360 E3-2	
P_n=	965.9	kips		
ΦP_n=	**869.3**	**kips**		**> Pu, OK**

Flexural Strength

Compact?	yes			
unbraced length, L_b=	1	ft		
c=	1		(doubly symm. I-shape)	AISC 360-10 F2-8a
L_p=	7.31	ft		AISC 360-10 F2-5
L_r=	20.76	ft	*lateral torsional bucklin*	AISC 360-10 F2-6
M_n=M_p=	1016.667	kip-ft	AISC 360-10 F2-1	
Φ=	0.9			
ΦM_n=	**915**	**kip-ft**		**> Mu, OK**

Combined Strength

@ Mid span			
P_r/P_c=	0.02	*< .2, USE H1-1b*	
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$=	0.271		AISC 360 H1-1b
@ start of span			
P_r/P_c=	0.03	*< .2, USE H1-1b*	
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$=	0.017		AISC 360 H1-1b
			<1, OK

Shear

C_v=	1	(for webs of rolled I-shaped men	AISC 360-10 G2.1a
A_w=	12.28	in^2	
$V_u<\Phi V_n$			
V_n=$.6A_wF_yC_v$=	368.46	kips	AISC 360-10 G2-1
Φ=	1		
ΦV_n=	**368.5**	**kips**	**>Vu, OK**

Deflection

Δ=	**$5wL^4/(384EI)+PL^3/(28EI)$**		
Δ(dead limit)=L/600<.375"=	**0.375**	**in**	**(per AISC Design Guide 3)**
Δ(livel limit)=L/360<.5"=	**0.250**	**in**	**(per AISC Design Guide 3)**
Δ(total limit)=L/480<.625"=	**0.625**	**in**	**(per AISC Design Guide 3)**
Δ(dead)=	**0.321**	**in**	**< Limit, OK**
Δ(live)=	**0.060**	**in**	**< Limit, OK**
Δ(total))=	**0.382**	**in**	**< Limit, OK**

USE **27x84**

Pairs of Links Required

Link Capacity/SF=	10	kips	
Pairs of Links Required=	5.8	links	
Links Used=	6		
Spacing =	5.733333	ft	ok

Hybrid Type I - Girder Case - 3rd Floor Girder

Infill Spacing=	10 ft		
Infill Span=	30 ft		
Girder Span=	30 ft		
Wall dead load=	83 psf		
Wall Height=	10 ft		
Wall dead load=	--- ---	830	plf
Roof Dead Load=	82 psf	0	k/beam
Floor Dead Load=	78 psf	11.7	k/beam
Live Load=	80 psf	12.0	
Roof Live Load=	0 psf	0	
Seismic load above=	58 kips (horizontal)		
Seismic load at this floor=	47 kips (horizontal)		
Total Seismic Load=	105 kips (horizontal)		
Moment induced into panel, Mp=	580 k-ft		
Panel Section Modulus, S=	143472 in^4		
stress= Mp/S=	0.048511 ksi		
load=stress*thickness=	4.44 k/ft		
Design Spectral Response Acceleration, SDS=	0.119		
Redundancy factor=	1		
M=(wl2/8+Pa)*(appropriate factors)=			
V=(wl/2+P)*(appropriate factors)=			

Beam Loads		Moment	Shear
1)	1.4D=	294.53 k-ft	33.8 kips
2)	1.2D + 1.6L + .5Lr=	444.45 k-ft	29.0 kips
3)	1.2D + 1.6Lr + .5L=	312.45 k-ft	29.0 kips
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L$=	396.13 k-ft	62.0 kips (PER FEM)
5)	$(.9 - .2\,S_{DS})D + \rho Q_E$=	225.02 k-ft	43.5 kips (PER FEM)

Axial Loads		
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L$=	52.50 kips (max)
5)	$(.9 - .2\,S_{DS})D + \rho Q_E$=	52.50 kips (max)

Calculated (refrence shear, moment, and axial diagrams)

Largest Flexural Load:	@ mid span load combination 2
V_u=	0 kips
P_U=	26.25 kips
M_U=	444 k-ft
Largest Axial Load:	@ start of span load combination 4
V_u=	62 kips
P_U=	52.50 kips
M_U=	0 k-ft

Try	W30x90	AISC T 1-1
E=	29000 ksi	
F_y=	50 ksi	

Compressive Strength

K_x=	1 (pin-pin)	AISC TC-A-7.1

L_x= 1 ft
slenderness= 1.03

K_y= 1 (pin-pin) AISC TC-A-7.1
L_y= 10 ft
slenderness= 57.42 ***governs***

4.71*SQRT(E/F_y)= 113.43 USE E3-2

F_e= 86.82 ksi AISC 360 E3-4
F_{cr}= 39.29 ksi AISC 360 E3-2

P_n= 1033.3 kips
ΦP_n= 930.0 kips **> Pu, OK**

Flexural Strength

Compact? yes

unbraced length, L_b= 1 ft
c= 1 (doubly symm. I-shape) AISC 360-10 F2-8a

L_p= 7.38 ft AISC 360-10 F2-5
L_r= 20.90 ft *lateral torsional bucklin* AISC 360-10 F2-6

M_n=M_p= 1179.167 kip-ft AISC 360-10 F2-1
Φ= 0.9
ΦM_n= 1061.25 kip-ft **> Mu, OK**

Combined Strength

@ Mid span
P_r/P_c= 0.03 *< .2, USE H1-1b*
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$= 0.433 AISC 360 H1-1b
@ start of span
P_r/P_c= 0.06 *< .2, USE H1-1b*
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$= 0.028 AISC 360 H1-1b
<1, OK

Shear

C_v= 1 (for webs of rolled I-shaped men AISC 360-10 G2.1a
A_w= 13.87 in^2

$V_u<\Phi V_n$
$V_n=.6A_wF_yC_v$= 415.95 kips AISC 360-10 G2-1
Φ= 1
ΦV_n= 416.0 kips **>Vu, OK**

Deflection

Δ= **$5wL^4/(384EI)+PL^3/(28EI)$**
Δ(dead limit)=L/600<.375"= **0.375 in** **(per AISC Design Guide 3)**
Δ(livel limit)=L/360<.25"= **0.250 in** **(per AISC Design Guide 3)**
Δ(total limit)=L/480<.625"= **0.625 in** **(per AISC Design Guide 3)**

Δ(dead)= **0.331 in** **< Limit, OK**
Δ(live)= **0.191 in** **< Limit, OK**
Δ(total))= **0.522 in** **< Limit, OK**

USE **W30x90**

Pairs of Links Required

Link Capacity/SF= 10 kips

Pairs of Links Required= 10.5 links
Links Used= 11

Spacing = 2.866667 ft ok

Hybrid Type I - Girder Case - 2nd Floor Girder

Infill Spacing=	10	ft		
Infill Span=	30	ft		
Girder Span=	30	ft		
Wall dead load=	120	psf		
Wall Height=	10	ft		
Wall dead load=	---	---	1200	plf
Roof Dead Load=	82	psf	0	k/beam
Floor Dead Load=	78	psf	11.7	k/beam
Live Load=	80	psf	12.0	k/beam
Roof Live Load=	0	psf	0	k/beam
Seismic load above=	105	kips (horizontal)		
Seismic load at this floor=	23.5	kips (horizontal)		
Total Seismic Load=	128.5	kips (horizontal)		
Moment induced into panel, Mp=	1050	k-ft		
Panel Section Modulus, S=	143472	in^4		
stress= Mp/S=	0.087822	ksi		
load=stress*thickness=	8.04	k/ft		
Design Spectral Response Acceleration, SDS=	0.119			
Redundancy factor=	1			
M=(wl2/8+Pa)*(appropriate factors)=				
V=(wl/2+P)*(appropriate factors)=				

Beam Loads		Moment		Shear	
1)	1.4D=	352.80	k-ft	41.6	kips
2)	1.2D + 1.6L + .5Lr=	494.40	k-ft	35.6	kips
3)	1.2D + 1.6Lr + .5L=	362.40	k-ft	35.6	kips
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L=$	435.64	k-ft	80.0	kips (PER FEM)
5)	$(.9 - .2\,S_{DS})D + \rho Q_E=$	265.20	k-ft	61.5	kips (PER FEM)

Axial Loads			
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L=$	64.25	kips (max)
5)	$(.9 - .2\,S_{DS})D + \rho Q_E=$	64.25	kips (max)

Calculated (refrence shear, moment, and axial diagrams)

Largest Flexural Load:	@ mid span load combination 2	
V_u=	0	kips
P_U=	32.125	kips
M_U=	494	k-ft
Largest Axial Load:	@ start of span load combination 4	
V_u=	80	kips
P_U=	64.25	kips
M_U=	0	k-ft

Try	W30x99		AISC T 1-1
E=	29000	ksi	
F_y=	50	ksi	

Compressive Strength

K_x=	1	(pin-pin)	AISC TC-A-7.1

L_x=	1	ft		
slenderness=	1.03			
K_y=	1	(pin-pin)	AISC TC-A-7.1	
L_y=	10	ft		
slenderness=		57.14 ***governs***		
4.71*SQRT(E/F_y)=	113.43		USE E3-2	
F_e=	87.65	ksi	AISC 360 E3-4	
F_{cr}=	39.38	ksi	AISC 360 E3-2	
P_n=	1142.0	kips		
ΦP_n=	**1027.8**	**kips**		**> Pu, OK**

Flexural Strength

Compact?	yes			
unbraced length, L_b=	1	ft		
c=	1		(doubly symm. I-shape)	AISC 360-10 F2-8a
L_p=	7.42	ft		AISC 360-10 F2-5
L_r=	21.34	ft	*lateral torsional bucklin*	AISC 360-10 F2-6
M_n=M_p=	1300	kip-ft	AISC 360-10 F2-1	
Φ=	0.9			
ΦM_n=	**1170**	**kip-ft**		**> Mu, OK**

Combined Strength

@ Mid span			
P_r/P_c=		0.03 *< .2, USE H1-1b*	
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$=	0.438		AISC 360 H1-1b
@ start of span			
P_r/P_c=		0.06 *< .2, USE H1-1b*	
$P_r/2P_c+(M_{rx}/M_{cx}+M_{ry}/M_{cy})$=	0.031		AISC 360 H1-1b
			<1, OK

Shear

C_v=	1	(for webs of rolled I-shaped men	AISC 360-10 G2.1a
A_w=	15.44	in^2	
$V_u<\Phi V_n$			
$V_n=.6A_wF_yC_v$=	463.32	kips	AISC 360-10 G2-1
Φ=	1		
ΦV_n=	**463.3**	**kips**	**>Vu, OK**

Deflection

Δ=	**$5wL^4/(384EI)+PL^3/(28EI)$**		
Δ(dead limit)=L/600<.375"=	**0.375**	**in**	**(per AISC Design Guide 3)**
Δ(livel limit)=L/360<.25"=	**0.250**	**in**	**(per AISC Design Guide 3)**
Δ(total limit)=L/480<.625"=	**0.625**	**in**	**(per AISC Design Guide 3)**
Δ(dead)=	**0.357**	**in**	**< Limit, OK**
Δ(live)=	**0.173**	**in**	**< Limit, OK**
Δ(total))=	**0.530**	**in**	**< Limit, OK**
Δ(total(.6D+.7E))=	**0.217**	**in**	
		USE	**W30x99**

Pairs of Links Required

Link Capacity/SF=	10	kips	
Pairs of Links Required=	12.85	links	
Links Used=	13		
Spacing =	2.388889	ft	ok

HYBRID TYPE 1
Column Design

Tributary area=	450	ft^2			
Unbraced length x, L_{bx}=	13	ft			
Unbraced length x, L_{by}=	13	ft			
Wall dead load=	98	psf	83	psf	
Wall Height=	30	ft	30	ft	
Wall Length=	15	ft	15	ft	
Wall dead load=	---	---	81.45	K	
Floor Dead Load=	78	psf	70.2	K	
Roof Dead Load=	82	psf	36.9	K	
Live Load=	80	psf	72.0	K	
Reduction Factor=	1.00				(ASCE 7-10 EQ. 4.7-1)
Roof Live Load=	20	psf	5.4	K	
Reduction Factor=	0.60				(ASCE 7-10 EQ. 4.7-1)
Seismic Load, P_E=	47.9	kips (Verticle)			

Seismic Force 2nd Floor, F_1=	23.5	K
Story Height=	13	ft
Seismic Force 3rd Floor, F_2=	47	K
Story Height=	13	ft
Seismic Force Roof, F_3=	58	K
Story Height=	13	ft
Eccentricy of links, h_{cp}=	3.00	ft
Distance between columns, B=	30.00	ft

$$P_i = \frac{\sum_{j=i+1}^{n}\left(F_j \times h_j\right) + F_i \times h_{cp}}{B}$$

Design Spectral Response Acceleration, SDS=	0.119	
Redundancy factor=	1	

		Axial Load, Pu	
1)	1.4D=	264	K
2)	1.2D+1.6L+.5Lr=	344	K
3)	1.2D+1.6Lr+.5L=	271	K
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L$=	351	K
5)	$(.9 - .2\ S_{DS})D + \rho Q_E$=	213	K

Try W14x	48	AISC T 4-1
Try W12x	45	
Try W10x	48	

E=	29000	ksi
F_y=	50	ksi

Compressive Strength

K_x=	1	(pin-pin)	AISC TC-A-7.1
L_x=	13	ft	
slenderness=	30.29		
K_y=	1	(pin-pin)	AISC TC-A-7.1
L_y=	13	ft	
slenderness=	80.00	***governs***	
4.71*SQRT(E/F_y)=	113.43		USE E3-2

F_e=	44.72	ksi	AISC 360 E3-4
F_{cr}=	31.31	ksi	AISC 360 E3-2
P_n=	410.2	kips	
ΦP_n=	**369.2**	**kips**	**> Pu, OK**

USE **W12X48**

Shear Wall Panel
Type: Hybrid Type I
Location: 2nd-3rd

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	44	psf (grouted @ 48" SPA.)

Panel Loads

Panel Dead Load =	12.61	kips
Other Dead Load=	0	kips
Total Dead Load=	12.61	kips
Load Factor =	0.90	
Pu=	**11.4**	**kips**
Shear Load =	58.00	Kips
Moment, M= V*h=	580.00	k-ft
Load Factor =	1.00	(seismic)
Mu=	**580**	**k-ft**
Shear load, V=	58.00	
Load Factor =	1.00	(seismic)
Vu=	**58.00**	

Check

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

S=	150420.309	**in^3**		
fr=	**103.00**	**psi**		
Mcr=Sfr=	**1291.11**	**k-ft**		
Mu/(Vudv)=	0.353		interpulate between eq	
factor=	5.73		(ACI 530-11 3-21&22)	
Vn=	6981	kips (max)		
Vnm=	346	kips	(ACI 530-11 3-23)	
Vns=	66		(ACI 530-11 3-24)	
Φ=	0.8			
ΦVn=	**329.9**			**ΦVn>Vu, OK**

Interaction Diagram

Material= conc
True thickness, t_{sp}= 7.625 in
d= 340.04 in
A_n= 2623.305 in^2

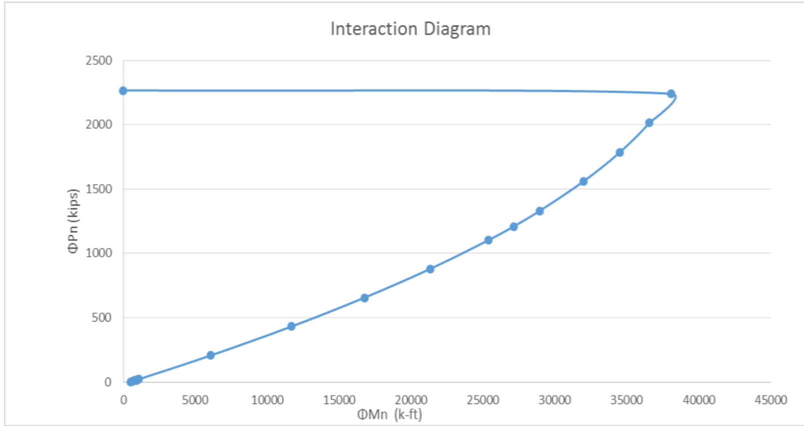
f'm= 1500 psi
f'r= psi
E'm= 1350000 psi
ε_{mu}= 0.0025

Bar= #5
A_b= 0.31 in^2
Spacing= 48 in o.c.
Area= 0.31 in^2

fy= 60000 psi
Es= 29000000 psi
ε_s= 0.00206897

c_{bal}= 186.059623
$(c/d)_{bal}$= 0.54716981

Φ_{ten}= 0.9
Φ_{comp}= 0.9



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (in-lb)	ΦMn (k-ft)	ΦPn (lb)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}$*d/(c-1)	=fs*As	$=\Phi C_{mu}$(d-.8c/2)		$=\Phi(C_{mu}$-Ts)	
Pure Compression						0	0	2266535.52	2267
	1	340.040	2489092.8	0	0	457051202	38088	2240184	2240
	0.9	306.036	2240183.52	8056	2497	438769154	36564	2013918	2014
	0.8	272.032	1991274.24	18125	5619	414393090	34533	1787090	1787
	0.7	238.028	1742364.96	31071	9632	383923010	31994	1559460	1559
	0.6	204.024	1493455.68	48333	14983	347358914	28947	1330625	1331
	0.54716981	186.060	1361956.438	60000	18600	325581864	27132	1209021	1209
	0.5	170.020	1244546.4	60000	18600	304700802	25392	1103352	1103
	0.4	136.016	995637.12	60000	18600	255948673	21329	879333	879
	0.3	102.012	746727.84	60000	18600	201102529	16759	655315	655
	0.2	68.008	497818.56	60000	18600	140162369	11680	431297	431
	0.1	34.004	248909.28	60000	18600	73128192	6094	207278	207
	0.017	5.781	42314.5776	60000	18600	12861726	1072	21343	21
	0.016	5.441	39825.4848	60000	18600	12110029	1009	19103	19.1
	0.0155	5.271	38580.9384	60000	18600	11733952	978	17983	18.0
	0.015	5.101	37336.392	60000	18600	11357722	946	16863	16.9
	0.014	4.761	34847.2992	60000	18600	10604807	884	14623	14.6
	0.013	4.421	32358.2064	60000	18600	9851282	821	12382	12
	0.012	4.080	29869.1136	60000	18600	9097147	758	10142	10
	0.01	3.400	24890.928	60000	18600	7587050	632	5662	6
	0.008	2.720	19912.7424	60000	18600	6074515	506	1181	1

Shear Wall Panel
Type: Hybrid Type I
Location: 2nd-3rd

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	81	psf (fully grouted)

Loads

Panel Dead Load =	23.22	kips
Other Dead Load=	0	kips
Total Dead Load=	23.22	kips
Load Factor =	0.90	
Pu=	**20.9**	**kips**
Shear Load =	105.00	Kips
Moment, M= V*h=	1050.00	k-ft
Load Factor =	1.00	(seismic)
Mu=	**1050**	**k-ft**
Shear load, V=	105.00	
Load Factor =	1.00	(seismic)
Vu=	**105.00**	

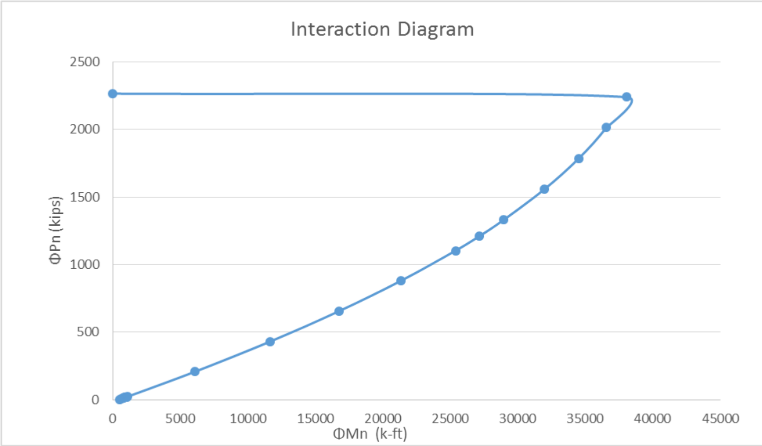
Check

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

Mu/(Vudv)=	0.353	interpulate between eq	
factor=	5.73	(ACI 530-11 3-21&22)	
Vn=	6981 kips (max)		
Vnm=	349 kips	(ACI 530-11 3-23)	
Vns=	198	(ACI 530-11 3-24)	
Φ=	0.8		
ΦVn=	**437.2227**		**ΦVn>Vu, OK**

Interaction Diagram

Material=	conc	
True thickness, t_{sp}=	7.625	in
d=	340.04	in
A_n=	2623.305	in^2
f'm=	1500	psi
E'm=	1350000	psi
ε_{mu}=	0.0025	
Bar=	#5	
A_b=	0.31	in^2
Spacing=	16	in o.c.
Area=	0.31	in^2
fy=	60000	psi
Es=	29000000	psi
ε_s=	0.002069	
c_{bal}=	186.0596	
$(c/d)_{bal}$=	0.54717	
Φ_{ten}=	0.9	
Φ_{comp}=	0.9	



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (k-ft)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}$*d/(c-1)	=fs*As	$=\Phi C_{mu}$(d-.8c/2)	$=\Phi(C_{mu}$-Ts)
Pure Compression						0	2267
	1	340.040	2489092.8	0	0	38088	2240
	0.9	306.036	2240183.52	8056	2497	36564	2014
	0.8	272.032	1991274.24	18125	5619	34533	1787
	0.7	238.028	1742364.96	31071	9632	31994	1559
	0.6	204.024	1493455.68	48333	14983	28947	1331
	0.54717	186.060	1361956.438	60000	18600	27132	1209
	0.5	170.020	1244546.4	60000	18600	25392	1103
	0.4	136.016	995637.12	60000	18600	21329	879
	0.3	102.012	746727.84	60000	18600	16759	655
	0.2	68.008	497818.56	60000	18600	11680	431
	0.1	34.004	248909.28	60000	18600	6094	207
	0.017	5.781	42314.5776	60000	18600	1072	21
	0.016	5.441	39825.4848	60000	18600	1009	19.1
	0.0155	5.271	38580.9384	60000	18600	978	18.0
	0.015	5.101	37336.392	60000	18600	946	16.9
	0.0145	4.931	36091.8456	60000	18600	915	15.7
	0.014	4.761	34847.2992	60000	18600	884	14.6
	0.013	4.421	32358.2064	60000	18600	821	12
	0.012	4.080	29869.1136	60000	18600	758	10
	0.01	3.400	24890.928	60000	18600	632	6
	0.008	2.720	19912.7424	60000	18600	506	1

Shear Wall Panel
Type: Hybrid Type I
Location: Ground Floor

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	120	psf (fully grouted 8" CMU w/ veneer)

Loads

Panel Dead Load =	34.40	kips
Other Dead Load=	0	kips
Total Dead Load=	34.40	kips
Load Factor =	0.90	
Pu=	**31.0**	**kips**
Shear Load =	128.00	Kips
Moment, M= V*h=	1280.00	k-ft
Load Factor =	1.00	(seismic)
Mu=	**1280**	**k-ft**
Shear load, V=	128.00	
Load Factor =	1.00	(seismic)
Vu=	**128.00**	

Check

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

Mu/(Vudv)=	0.353		interpulate between eq
factor=	5.73		(ACI 530-11 3-21&22)
Vn=	6981	kips (max)	
Vnm=	351	kips	(ACI 530-11 3-23)
Vns=	198		(ACI 530-11 3-24)
Φ=	0.8		
ΦVn=	**439.2353**		**ΦVn>Vu, OK**

Interaction Diagram

Material= conc
True thickness, t_{sp}= 7.625 in
d= 340.04 in
A_n= 2623.305 in^2

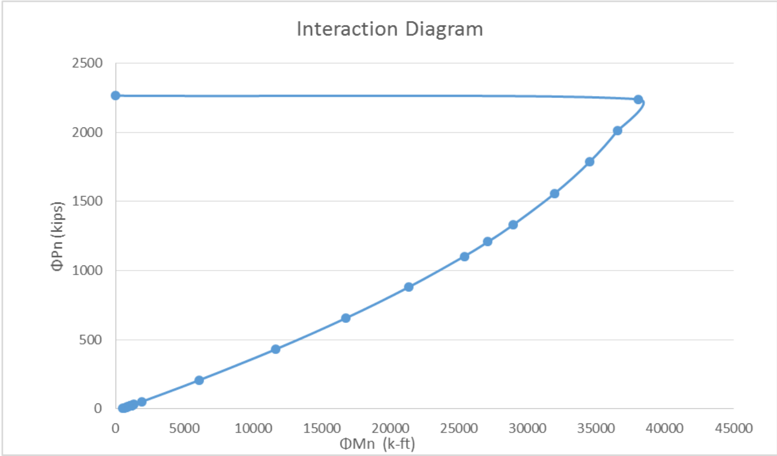
f'm= 1500 psi
E'm= 1350000 psi
ε_{mu}= 0.0025

Bar= #5
A_b= 0.31 in^2
Spacing= 16 in o.c.
Area= 0.31 in^2

fy= 60000 psi
Es= 29000000 psi
ε_s= 0.002069

c_{bal}= 186.0596
$(c/d)_{bal}$= 0.54717

Φ_{ten}= 0.9
Φ_{comp}= 0.9



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (k-ft)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}$*d/(c-1)	=fs*As	$=\Phi C_{mu}$(d-.8c/2)	$=\Phi(C_{mu}$-Ts)
Pure Compression						0	2267
	1	340.040	2489092.8	0	0	38088	2240
	0.9	306.036	2240183.52	8056	2497	36564	2014
	0.8	272.032	1991274.24	18125	5619	34533	1787
	0.7	238.028	1742364.96	31071	9632	31994	1559
	0.6	204.024	1493455.68	48333	14983	28947	1331
	0.54717	186.060	1361956.438	60000	18600	27132	1209
	0.5	170.020	1244546.4	60000	18600	25392	1103
	0.4	136.016	995637.12	60000	18600	21329	879
	0.3	102.012	746727.84	60000	18600	16759	655
	0.2	68.008	497818.56	60000	18600	11680	431
	0.1	34.004	248909.28	60000	18600	6094	207
	0.03	10.201	74672.784	60000	18600	1882	50
	0.021	7.141	52270.9488	60000	18600	1322	30
	0.02	6.801	49781.856	60000	18600	1259	28
	0.019	6.461	47292.7632	60000	18600	1197	26
	0.0181	6.155	45052.57968	60000	18600	1141	23.8
	0.018	6.121	44803.6704	60000	18600	1134	23.6
	0.0179	6.087	44554.76112	60000	18600	1128	23.4
	0.0178	6.053	44305.85184	60000	18600	1122	23.1
	0.0177	6.019	44056.94256	60000	18600	1116	22.9
	0.0176	5.985	43808.03328	60000	18600	1109	22.7
	0.0175	5.951	43559.124	60000	18600	1103	22
	0.017	5.781	42314.5776	60000	18600	1072	21
	0.016	5.441	39825.4848	60000	18600	1009	19
	0.015	5.101	37336.392	60000	18600	946	17
	0.014	4.761	34847.2992	60000	18600	884	15
	0.013	4.421	32358.2064	60000	18600	821	12
	0.012	4.080	29869.1136	60000	18600	758	10
	0.01	3.400	24890.928	60000	18600	632	6
	0.008	2.720	19912.7424	60000	18600	506	1

Appendix D - Sample Design for a Type II Hybrid Masonry Frame

Hybrid Type II - Girder Case - Roof Girder

Infill Spacing=	10	ft		
Infill Span=	30	ft		
Girder Span=	30	ft		
Wall Height (above)=	4	ft		

Pre Wall Phase Loads

Roof Dead Load=	82	psf	12.3	k/beam
Floor Dead Load=	0	psf	0	k/beam
Construction Live Load=	20	psf	2.7	k/beam reduced

M=(Pa)*(appropriate factors)=

V=(P)*(appropriate factors)=

Beam Load		Moment		Shear	
1)	1.4D=	172.20	k-ft	17.22	kips
2)	1.2D + 1.6L + .5Lr=	161.10	k-ft	16.11	kips
3)	1.2D + 1.6Lr + .5L=	190.80	k-ft	19.08	kips

Post Wall Loads - (distributed to masonry wall)

Wall dead load=	83	psf		
Wall dead load=	---	---	332	plf
Live Load=	0	psf	0	k/beam reduced
Roof Live Load=	20	psf	2.7	k/beam reduced

Design Spectral Response Acceleration, SDS=	0.119	
Redundancy factor=	1	
Total horizontal load (factored)=	58	kips

Construction Phase Design

Maximum loads:

V_u=	19	kips @ ends
P_U=	29.0	kips @ ends
M_U=	191	k-ft @ mid span

Try	21x55		AISC T 1-1
E=	29000	ksi	
F_y=	50	ksi	

Flexural Strength

Compact?	yes			
unbraced length, L_b=	10	ft		
c=	1		(doubly symm. I-shape)	AISC 360-10 F2-8a
L_p=	6.11	ft		AISC 360-10 F2-5
L_r=	31.03	ft	*L-T buckling applies, USE F2-2*	
				AISC 360-10 F2-6
M_n=M_p=	525	kip-ft		AISC 360-10 F2-1
$M_n=C_b(M_p-(F_yZ_x-.7F_yS_x)(L_b-L_p)/(L_r-L_p))$=	517.8471	kip-ft		AISC 360-10 F2-2
				AISC 360-10 F2-3
Φ=	0.9			
ΦM_n=	**466.06**	**kip-ft**		**> Mu, OK**

Shear

C_v=	1	(for webs of rolled I-shaped members)	AISC 360-10 G2.1a
A_w=	7.80	in^2	
V_u<ΦV_n			
V_n=.6$A_w F_y C_v$=	255	kips	AISC Table 3-2
Φ=	1		
ΦV_n=	**255.0**	**kips**	**>Vu, OK**

Deflection

Δ=	**$PL^3/(28EI)$**		
Δ(live limit)=L/240=	**1.500**	**in**	
Δ(total limit)=L/180=	**2.000**	**in**	
Δ(dead)=	**0.620**	**in**	
Δ(live)=	**0.136**	**in**	**< Limit, OK**
Δ(total))=	**0.756**	**in**	**< Limit, OK**

USE **21x55**

Shear studs required

diameter=	1	in	
conc. Strength, f'c=	3000	psi	
crossectional area, A_{sa}=	0.785398	in^2	
Ec=w_c^1.5sqrt(f'c)=	21488.31	psi	
Fu=	65	ksi	
Rg=	1		AISC 360 I8
Rp=	0.75		AISC 360 I8
Nominal Stud Strength, Qn=	=.5A_{sa}sqrt($f'_c E_c$)< $R_g R_p A_{sa} F_u$		AISC 360 Eq I8-1
Qn=	3.15	kips	
Φ=	0.65		AISC 360 I3a
ΦV_n=	2.05	kips per stud	
Studs required=	29		

Hybrid Type II - Girder Case - 2nd & 3rd Floor Girder

Infill Spacing=	10	ft
Infill Span=	30	ft
Girder Span=	30	ft
Wall Height (above)=	10	ft

Pre Wall Phase Loads

Roof Dead Load=	0	psf	0	k/beam
Floor Dead Load=	78	psf	11.7	k/beam
Construction Live Load=	20	psf	2.7	k/beam reduced

M=(Pa)*(appropriate factors)=
V=(P)*(appropriate factors)=

Beam Load		Moment		Shear	
1)	1.4D=	163.80	k-ft	16.38	kips
2)	1.2D + 1.6L + .5Lr=	153.90	k-ft	15.39	kips
3)	1.2D + 1.6Lr + .5L=	183.60	k-ft	18.36	kips

Post Wall Loads - (distributed to masonry wall)

Wall dead load=	83	psf		
Wall dead load=	---	---	830	plf
Live Load=	80	psf	10.8	k/beam reduced
Floor Live Load=	20	psf	2.7	k/beam reduced

Design Spectral Response Acceleration, SDS=	0.119		
Redundancy factor=	1		
Total horizontal load at 3rd Floor (factored)=	105	kips	
Total horizontal load at 2nd Floor (factored)=	128.6	kips	Governs

Construction Phase Design

Maximum loads:

V_u=	18	kips @ ends
P_U=	64.3	kips @ ends
M_U=	184	k-ft @ mid span

Try	21x55		AISC T 1-1
E=	29000	ksi	
F_y=	50	ksi	

Flexural Strength

Compact?	yes			
unbraced length, L_b=	10	ft		
c=	1		(doubly symm. I-shape)	AISC 360-10 F2-8a
L_p=	4.59	ft		AISC 360-10 F2-5
L_r=	13.59	ft	*L-T buckling applies, USE F2-2*	
				AISC 360-10 F2-6
M_n=M_p=	458.3333	kip-ft		AISC 360-10 F2-1
$M_n=C_b(M_p-(F_yZ_x-.7F_yS_x)(L_b-L_p)/(L_r-L_p))$=	221.6872	kip-ft		AISC 360-10 F2-2
				AISC 360-10 F2-3
Φ=	0.9			
ΦM_n=	**199.52**	**kip-ft**		**> Mu, OK**

Hybrid Type II - Column
columns

Tributary area=	450	ft^2		
Unbraced length x, L_{bx}=	12	ft		
Unbraced length x, L_{by}=	12	ft		
Wall dead load=	83	psf		83 psf
Wall Height=	30	ft		30 ft
Wall Length=	15	ft		15 ft
Wall dead load=	---	---	0	K (supported by masonry panels)
Floor Dead Load=	78	psf	70.2	K
Roof Dead Load=	82	psf	36.9	K
Live Load=	0	psf	0.0	K (supported by masonry panels)
Reduction Factor=	0.50			(ASCE 7-10 EQ. 4.7-1)
Roof Live Load=	0	psf	0.0	K (supported by masonry panels)
Reduction Factor=	0.60			(ASCE 7-10 EQ. 4.7-1)
Seismic Load, P_E=	46.3	kips (Verticle)		

$$P_i = \frac{\sum_{j=i+1}^{n}(F_j \times h_j) + F_i \times h_{cp}}{B}$$

Seismic Force 2nd Floor, F_1=	23.5	K
Story Height=	13	ft
Seismic Force 3rd Floor, F_2=	47	K
Story Height=	13	ft
Seismic Force Roof, F_3=	58	K
Story Height=	13	ft
Eccentricity of connections, h_{cp}=	1.00	ft
Distance between columns, B=	30.00	ft
Design Spectral Response Acceleration, SDS=	0.119	
Redundancy factor=	1	

		Axial Load, Pu	
1)	1.4D=	150	K
2)	1.2D+1.6L+.5Lr=	129	K
3)	1.2D+1.6Lr+.5L=	129	K
4)	$(1.2 + .2S_{DS})D + \rho Q_E + L$=	177	K
5)	$(.9 - .2\ S_{DS})D + \rho Q_E$=	140	K

Try W14x	43	
Try W12x	40	AISC T 4-1
Try W10x	33	
Try W8x	31	
E=	29000	ksi
F_y=	50	ksi

Compressive Strength

K_x=	1	(pin-pin)	AISC TC-A-7.1
L_x=	13	ft	
slenderness=	44.96		

K_y=	1	(pin-pin)	AISC TC-A-7.1
L_y=	13	ft	
slenderness=	77.23	***governs***	
4.71*SQRT(E/F_y)=	113.43	USE E3-2	
F_e=	47.99	ksi	AISC 360 E3-4
F_{cr}=	32.33	ksi	AISC 360 E3-2
P_n=	295.2	kips	
ΦP_n=	**265.6**	**kips**	**> Pu, OK**

USE **W8X31**

Shear Wall Panel
Type: Hybrid Type I
Location: 3rd -Roof

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	44	psf (8" CMU grouted @ 48" SPA.)

Panel Loads

Girder Case

Tracked dead load=	9.96	kips (parapet)
Panel Dead Load =	12.6	kips
Roof Dead Load=	24.6	kips
Veneer Dead Load=	11.2	kips
Total Dead Load=	48.40	kips
Roof Live load =	5.40	kips (reduced)
Live Load =	0.00	kips (reduced)

Beam Case

Tracked dead load=	9.96	kips (parapet)
Panel Dead Load =	12.6	kips
Roof Dead Load=	12.3	kips
Veneer Dead Load=	11.2	kips
Total Dead Load=	36.10	kips
Roof Live load =	3.00	kips (reduced)
Live Load =	0.00	kips (reduced)
Shear Load (seismic) =	58.00	kips

Axial Check - 1.2D+1.6(L or Lr) with girder case governs

Pu=	**66.7**	**kips**	
radius of giration, r=	2.2	in	
h/r=	54.5	<99 USE EQ. 3-18	
Pn=	2136	kips	(ACI 530-11 EQ. 3-18)
Φ=	0.9		
ΦPn=	**1923**	**kips**	**ΦPn>Pu, OK**

Overturning Check - 0.9D+1.0E with Beam Case Governs

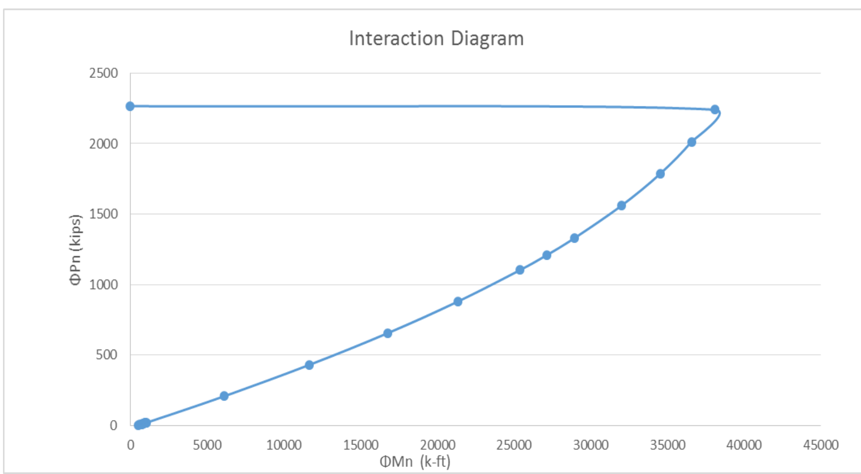
Load Factor =	0.90	
Pu=	**32.5**	**kips**
Moment, M= V*h=	580.00	k-ft
Load Factor =	1.00	(seismic)
Mu=	**580**	**k-ft**
Shear load, V=	58.00	
Load Factor =	1.00	(seismic)
Vu=	**58.00**	**kips**
S=	150420.3	**in^3**
fr=	**103.00**	**psi**
Mcr=Sfr=	**1291.11**	**k-in**

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

Mu/(Vudv)=	0.353		interpulate between eq
factor=	5.73		(ACI 530-11 3-21&22)
Vn=	582	kips (max)	
Vnm=	352	kips	(ACI 530-11 3-23)
Vns=	66		(ACI 530-11 3-24)
Φ=	0.8		
ΦVn=	**334.1**	**kips**	**ΦVn>Vu, OK**

Interaction Diagram

Material=	conc	
True thickness, t_{sp}=	7.625	in
d=	340.04	in
A_n=	2623.305	in^2
f'm=	1500	psi
E'm=	1350000	psi
ε_{mu}=	0.0025	
Bar=	#5	
A_b=	0.31	in^2
Spacing=	48	in o.c.
Area=	0.31	in^2
fy=	60000	psi
Es=	29000000	psi
ε_s=	0.002069	
c_{bal}=	186.0596	
$(c/d)_{bal}$=	0.54717	
Φ_{ten}=	0.9	
Φ_{comp}=	0.9	



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (in-lb)	ΦMn (k-ft)	ΦPn (lb)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}*d/(c-1)$	=fs*As	$=\Phi C_{mu}(d-.8c/2)$		$=\Phi(C_{mu}-Ts)$	
Pure Compression						0	0	2266535.52	2267
	1	340.040	2489092.8	0	0	457051202	38088	2240184	2240
	0.9	306.036	2240183.52	8056	2497	438769154	36564	2013918	2014
	0.8	272.032	1991274.24	18125	5619	414393090	34533	1787090	1787
	0.7	238.028	1742364.96	31071	9632	383923010	31994	1559460	1559
	0.6	204.024	1493455.68	48333	14983	347358914	28947	1330625	1331
	0.54717	186.060	1361956.438	60000	18600	325581864	27132	1209021	1209
	0.5	170.020	1244546.4	60000	18600	304700802	25392	1103352	1103
	0.4	136.016	995637.12	60000	18600	255948673	21329	879333	879
	0.3	102.012	746727.84	60000	18600	201102529	16759	655315	655
	0.2	68.008	497818.56	60000	18600	140162369	11680	431297	431
	0.1	34.004	248909.28	60000	18600	73128192	6094	207278	207
	0.017	5.781	42314.5776	60000	18600	12861726	1072	21343	21
	0.016	5.441	39825.4848	60000	18600	12110029	1009	19103	19.1
	0.0155	5.271	38580.9384	60000	18600	11733952	978	17983	18.0
	0.015	5.101	37336.392	60000	18600	11357722	946	16863	16.9
	0.014	4.761	34847.2992	60000	18600	10604807	884	14623	14.6
	0.013	4.421	32358.2064	60000	18600	9851282	821	12382	12
	0.012	4.080	29869.1136	60000	18600	9097147	758	10142	10
	0.01	3.400	24890.928	60000	18600	7587050	632	5662	6
	0.008	2.720	19912.7424	60000	18600	6074515	506	1181	1

Shear Wall Panel
Type: Hybrid Type I
Location: 2nd-3rd

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	44	psf (8" CMU grouted @ 48" SPA.)

Panel Loads

Girder Case

Tracked Dead Load=	48.40	kips
Panel Dead Load =	12.61	kips
Floor Dead Load=	23.4	kips
Total Dead Load=	84.41	kips
Roof Live load =	5.40	kips (reduced)
Live Load =	46.80	kips (reduced)

Beam Case

Tracked Dead Load=	36.10	kips
Panel Dead Load =	12.61	kips
Floor Dead Load=	10.8	kips
Total Dead Load=	59.51	kips
Roof Live load =	3.00	kips
Live Load =	24.00	kips
Shear Load (seismic) =	105.00	kips

Axial Check - 1.2D+1.6(L or Lr) with girder case governs

Pu=	**176.2**	**kips**		
radius of giration, r=	2.2	in		
h/r=	54.5	<99 USE EQ. 3-18		
Pn=	2136	kips	(ACI 530-11 EQ. 3-18)	
Φ=	0.9			
ΦPn=	**1923**	**kips**		**ΦPn>Pu, OK**

Overturning Check - 0.9D+1.0E with Beam Case Governs

Load Factor =	0.90			
Pu=	**53.6**	**kips**		
Moment, M= V*h=	1050.00	k-ft		
Load Factor =	1.00	(seismic)		
Mu=	**1050**	**k-ft**		
Shear load, V=	105.00			
Load Factor =	1.00	(seismic)		
Vu=	**105.00**	**kips**		
S=	150420.3	**in^3**		
fr=	**103.00**	**psi**		
Mcr=Sfr=	**1291.11**	**k-in**		

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

Mu/(Vudv)=	0.353		interpulate between eq	
factor=	5.73		(ACI 530-11 3-21&22)	
Vn=	582	kips (max)		
Vnm=	357	kips	(ACI 530-11 3-23)	
Vns=	66		(ACI 530-11 3-24)	
Φ=	0.8			
ΦVn=	**338.3**	**kips**		**ΦVn>Vu, OK**

Interaction Diagram

Material= conc
True thickness, t_{sp}= 7.625 in
d= 340.04 in
A_n= 2623.305 in^2

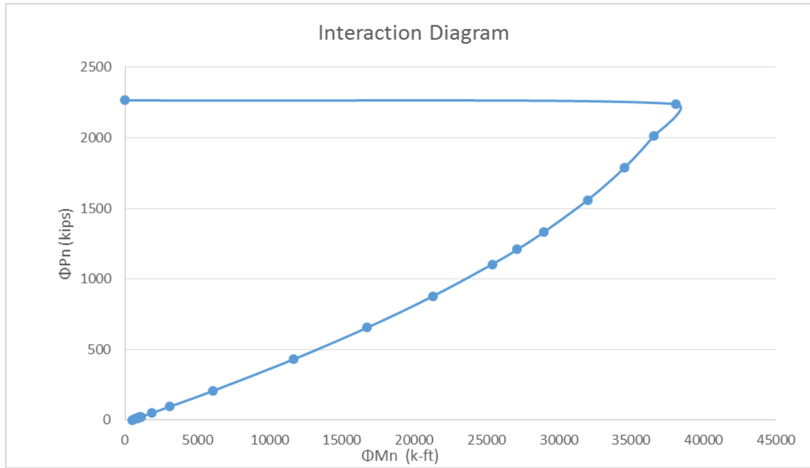
f'm= 1500 psi
E'm= 1350000 psi
ε_{mu}= 0.0025

Bar= #5
A_b= 0.31 in^2
Spacing= 48 in o.c.
Area= 0.31 in^2

fy= 60000 psi
Es= 29000000 psi
ε_s= 0.002069

c_{bal}= 186.0596
$(c/d)_{bal}$= 0.54717

Φ_{ten}= 0.9
Φ_{comp}= 0.9



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (k-ft)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}$*d/(c-1)	=fs*As	$=\Phi C_{mu}$(d-.8c/2)	$=\Phi(C_{mu}$-Ts)
Pure Compression						0	2267
	1	340.040	2489092.8	0	0	38088	2240
	0.9	306.036	2240183.52	8056	2497	36564	2014
	0.8	272.032	1991274.24	18125	5619	34533	1787
	0.7	238.028	1742364.96	31071	9632	31994	1559
	0.6	204.024	1493455.68	48333	14983	28947	1331
	0.54717	186.060	1361956.438	60000	18600	27132	1209
	0.5	170.020	1244546.4	60000	18600	25392	1103
	0.4	136.016	995637.12	60000	18600	21329	879
	0.3	102.012	746727.84	60000	18600	16759	655
	0.2	68.008	497818.56	60000	18600	11680	431
	0.1	34.004	248909.28	60000	18600	6094	207
	0.05	17.002	124454.64	60000	18600	3110	95
	0.03	10.201	74672.784	60000	18600	1882	50
	0.018	6.121	44803.6704	60000	18600	1134	24
	0.017	5.781	42314.5776	60000	18600	1072	21
	0.0165	5.611	41070.0312	60000	18600	1040	20
	0.016	5.441	39825.4848	60000	18600	1009	19.1
	0.0155	5.271	38580.9384	60000	18600	978	18.0
	0.015	5.101	37336.392	60000	18600	946	16.9
	0.014	4.761	34847.2992	60000	18600	884	14.6
	0.013	4.421	32358.2064	60000	18600	821	12
	0.012	4.080	29869.1136	60000	18600	758	10
	0.01	3.400	24890.928	60000	18600	632	6
	0.008	2.720	19912.7424	60000	18600	506	1

Shear Wall Panel
Type: Hybrid Type II
Location: 1st to 2nd

Properties

Panel Height, h =	10	ft
Panel Length, L =	28.67	ft
Panel weight =	44	psf (8" CMU grouted @ 48" SPA.)

Panel Loads

Girder Case

Tracked Dead Load=	59.5	kips
Panel Dead Load =	12.61	kips
Floor Dead Load=	23.4	kips
Total Dead Load=	95.53	kips
Roof Live load =	2.70	kips (reduced)
Live Load =	43.20	kips (reduced)

Beam Case

Tracked Dead Load=	59.51	kips
Panel Dead Load =	12.61	kips
Floor Dead Load=	10.8	kips
Total Dead Load=	82.93	kips
Roof Live load =	3.00	kips
Live Load =	36.00	kips
Shear Load (seismic) =	128.60	kips

Axial Check - 1.2D+1.6(L or Lr) with girder case governs

Pu=	**183.8**	**kips**		
radius of giration, r=	2.2	in		
h/r=	54.5	<99 USE EQ. 3-18		
Pn=	2136	kips	(ACI 530-11 EQ. 3-18)	
Φ=	0.9			
ΦPn=	**1923**	**kips**		**ΦPn>Pu, OK**

Overturning Check - 0.9D+1.0E with Beam Case Governs

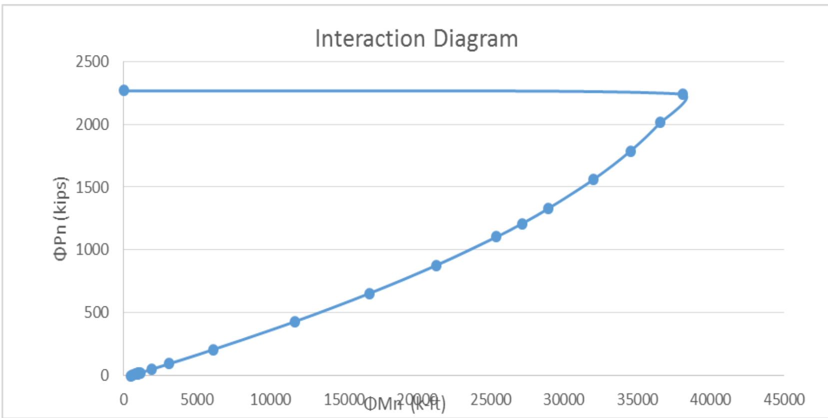
Load Factor =	0.90	
Pu=	**74.6**	**kips**
Moment, M= V*h=	1286.00	k-ft
Load Factor =	1.00	(seismic)
Mu=	**1286**	**k-ft**
Shear load, V=	128.60	
Load Factor =	1.00	(seismic)
Vu=	**128.60**	**kips**
S=	150420.3	**in^3**
fr=	**103.00**	**psi**
Mcr=Sfr=	**1291.11**	**k-in**

The combination of the moment and axial load lies within the bounds of the interation diagram below and is therefore a valid design.

Mu/(Vudv)=	0.353		interpulate between eq
factor=	5.73		(ACI 530-11 3-21&22)
Vn=	582	kips (max)	
Vnm=	362	kips	(ACI 530-11 3-23)
Vns=	66		(ACI 530-11 3-24)
Φ=	0.8		
ΦVn=	**342.6**	**kips**	**ΦVn>Vu, OK**

Interaction Diagram

Material=	conc	
True thickness, t_{sp}=	7.625	in
d=	340.04	in
A_n=	2623.305	in^2
f'm=	1500	psi
E'm=	1350000	psi
ε_{mu}=	0.0025	
Bar=	#5	
A_b=	0.31	in^2
Spacing=	48	in o.c.
Area=	0.31	in^2
fy=	60000	psi
Es=	29000000	psi
ε_s=	0.002069	
c_{bal}=	186.0596	
$(c/d)_{bal}$=	0.54717	
Φ_{ten}=	0.9	
Φ_{comp}=	0.9	



	c/d	c (in)	Cmasonry (lb)	fs (psi)	Tsteel (lb)	ΦMn (in-lb)	ΦMn (k-ft)	ΦPn (lb)	ΦPn (k)
		=c/d*d	=0.8f'm*0.8ct	$=E_s\varepsilon_{mu}*d/(c-1)$	=fs*As	$=\Phi C_{mu}(d-.8c/2)$		$=\Phi(C_{mu}-Ts)$	
Pure Compression						0	0	2266535.52	2267
	1	340.040	2489092.8	0	0	457051202	38088	2240184	2240
	0.9	306.036	2240183.52	8056	2497	438769154	36564	2013918	2014
	0.8	272.032	1991274.24	18125	5619	414393090	34533	1787090	1787
	0.7	238.028	1742364.96	31071	9632	383923010	31994	1559460	1559
	0.6	204.024	1493455.68	48333	14983	347358914	28947	1330625	1331
	0.54717	186.060	1361956.438	60000	18600	325581864	27132	1209021	1209
	0.5	170.020	1244546.4	60000	18600	304700802	25392	1103352	1103
	0.4	136.016	995637.12	60000	18600	255948673	21329	879333	879
	0.3	102.012	746727.84	60000	18600	201102529	16759	655315	655
	0.2	68.008	497818.56	60000	18600	140162369	11680	431297	431
	0.1	34.004	248909.28	60000	18600	73128192	6094	207278	207
	0.05	17.002	124454.64	60000	18600	37325848	3110	95269	95
	0.03	10.201	74672.784	60000	18600	22578329	1882	50466	50
	0.018	6.121	44803.6704	60000	18600	13612813	1134	23583	24
	0.017	5.781	42314.5776	60000	18600	12861726	1072	21343	21
	0.0165	5.611	41070.0312	60000	18600	12485953	1040	20223	20
	0.016	5.441	39825.4848	60000	18600	12110029	1009	19103	19.1
	0.0155	5.271	38580.9384	60000	18600	11733952	978	17983	18.0
	0.015	5.101	37336.392	60000	18600	11357722	946	16863	16.9
	0.014	4.761	34847.2992	60000	18600	10604807	884	14623	14.6
	0.013	4.421	32358.2064	60000	18600	9851282	821	12382	12
	0.012	4.080	29869.1136	60000	18600	9097147	758	10142	10
	0.01	3.400	24890.928	60000	18600	7587050	632	5662	6
	0.008	2.720	19912.7424	60000	18600	6074515	506	1181	1

Appendix E - Economic Comparison

Summary

Lateral System		Structural Steel		Masonry		Concrete				Reinforcement		Connections							
												Bolts		Welds (shop)		Welds (field)		Studs	Links
		tons	Improvement	Units	Improvement	Concrete (cy)	Improvement	Grout (cy)	Improvement	tons	Improvement	Quantity	Improvement	Length (in)	improvement	Length (in)	improvement	Quantity	Quantity
Moment Frame (standard)		111	0%	18240	0%	505	0%	38	0%	35	0%	1536	0%	31236	0%			0	0
												0							
Hybrid Masonry Type I		78	30%	18804	3%	608	20%	86	-127%	44	-28%	1776	-16%	3456	89%	424		0	240
												0							
Hybrid Masonry Type II		58	48%	19148	5%	392	22%	41	-7%	28	19%	480	69%	2880	91%			1800	0

Structural Steel

Total bays considered:				16	(governed by MF)									
Total columns considered:				20										
Bay span:				30	ft									
Individual Components	**Section**	**Depth**		**Weight (lb/ft)**	**Length (ft)**	**Quanity**	**Subtotal**							
Typical Gravity														
Column:	W	12	x	40	39	1	*1560.0*	*lbs*						
Roof Beam:	W	24	x	55	30	1	*1650.0*	*lbs*						
3rd Floor Beam:	W	24	x	84	30	1	*2520.0*	*lbs*						
2nd Floor Beam:	W	24	x	84	30	1	*2520.0*	*lbs*						
Roof Girder:	W	27	x	84	30	1	*2520.0*	*lbs*						
3rd Floor Girder:	W	30	x	90	30	1	*2700.0*	*lbs*	**Required Gravity Components to Balance compariso**					
2nd Floor Girder:	W	30	x	90	30	1	*2700.0*	*lbs*	**Weight (lb)**	**Quantitiy**	**Sub Total**		**Total**	
Typical Moment Frame													***93***	***tons***
Column:	W	27	x	129	39	20	*#######*	*lbs*	*1560.0*	0	*100620*	*lbs*	***0%***	***(standard***
Roof Beam:	W	21	x	55	30	8	*13200.0*	*lbs*	*1650.0*	0	*13200*	*lbs*		
3rd Floor Beam:	W	24	x	62	30	8	*14880.0*	*lbs*	*2520.0*	0	*14880*	*lbs*		
2nd Floor Beam:	W	24	x	62	30	8	*14880.0*	*lbs*	*2520.0*	0	*14880*	*lbs*		
Roof Girder:	W	21	x	55	30	8	*13200.0*	*lbs*	*2520.0*	0	*13200*	*lbs*		
3rd Floor Girder:	W	24	x	62	30	8	*14880.0*	*lbs*	*2700.0*	0	*14880*	*lbs*		
2nd Floor Girder:	W	24	x	62	30	8	*14880.0*	*lbs*	*2700.0*	0	*14880*	*lbs*		
Typical Hybrid Masonry Type I													***77***	***tons***
Column:	W	12	x	48	39	16	*29952.0*	*lbs*	1560.0	4	*36192*	*lbs*	***17%***	***decrease***
Roof Beam:	W	24	x	55	30	4	*6600.0*	*lbs*	1650.0	4	*13200*	*lbs*		
3rd Floor Beam:	W	24	x	84	30	4	*10080.0*	*lbs*	2520.0	4	*20160*	*lbs*		
2nd Floor Beam:	W	27	x	84	30	4	*10080.0*	*lbs*	2520.0	4	*20160*	*lbs*		
Roof Girder:	W	27	x	84	30	4	*10080.0*	*lbs*	2520.0	4	*20160*	*lbs*		
3rd Floor Girder:	W	30	x	90	30	4	*10800.0*	*lbs*	2700.0	4	*21600*	*lbs*		
2nd Floor Girder:	W	30	x	99	30	4	*11880.0*	*lbs*	2700.0	4	*22680*	*lbs*		
Typical Hybrid Masonry Type II													***57***	***tons***
Column:	W	8	x	31	39	16	*19344.0*	*lbs*	1560.0	4	*25584*	*lbs*	***38%***	***decrease***
Roof Beam:	W	16	x	31	30	4	*3720.0*	*lbs*	1650.0	4	*10320*	*lbs*		
3rd Floor Beam:	W	16	x	31	30	4	*3720.0*	*lbs*	2520.0	4	*13800*	*lbs*		
2nd Floor Beam:	W	16	x	31	30	4	*3720.0*	*lbs*	2520.0	4	*13800*	*lbs*		
Roof Girder:	W	21	x	55	30	4	*6600.0*	*lbs*	2520.0	4	*16680*	*lbs*		
3rd Floor Girder:	W	21	x	55	30	4	*6600.0*	*lbs*	2700.0	4	*17400*	*lbs*		
2nd Floor Girder:	W	21	x	55	30	4	*6600.0*	*lbs*	2700.0	4	*17400*	*lbs*		

Masonry

Masonry Panel Type	Condition	Masonry Panel Width (ft)	Masonry Panel Height (ft)	Masonry Panel Units	Grout Spacing (in)	Grout Columns	Grout Volume (yd^3)	Reinforcement Spacing (in)	Reinforcement Quantity	Reinforcement Diam. (in)	Reinforcement Volume (in^3)	Reinforcement Weight (lb)	Shear Studs Quantity	Hybrid Links Quantity	Reinf. Welds
Masonry Infill (between typical Gravity members)	Beam	28.67	32.2	1210	48	9	2.41	48	9	0.625	88.9	302	0	0	0
	Girder	28.67	31.0	1146	48	9	2.32	48	9	0.625	85.7	291	0	0.0	0
												0			
Masonry Infill (between typical Moment Frame members)	Beam	27.33	32.4	1140	48	8	2.16	48	8	0.625	79.6	271	0	*0.0*	0
	Girder	27.33	32.4	1140	48	8	2.16	48	8	0.625	79.6	271	0	*0.0*	0
											0.0	0			
Type I Hybrid Masonry	Beam	28.67	10.9	1210	48	9	1.00	48	9	0.625	30.0	102	0	6.0	9
		28.67	10.9		8	43	3.89	48	9	0.625	30.0	102	0	11.0	9
		28.67	10.2		8	43	3.64	48	9	0.625	28.1	95	0	13.0	9
	Girder	28.67	10.4	1135	48	9	0.97	48	9	0.625	28.6	97	0	6	9
		28.67	10.4		8	43	3.71	48	9	0.625	28.6	97	0	11	9
		28.67	9.9		8	43	3.55	48	9	0.625	27.4	93	0	13	9
Type II Hybrid Masonry	Beam	28.67	34.2	1242	48	9	2.75	48	9	0.625	94.3	321	225	0	0
	Girder	28.67	32.9	1189	48	9	2.65	48	9	0.625	90.9	309	225	0	0

Masonry System Comparison		Bay Count (L)	Bay Count (G)	Masonry Units (L)	Masonry Units (G)	Masonry Units Total	Grout (yd^3) (L)	Grout (yd^3) (G)	Grout (yd^3) Total	Reinforcement (lbs) (L)	Reinforcement (lbs) (G)	Reinforcement (lbs) Total	Shear Studs	Links	Reinf. Welds (in)
Moment Frame						*18240*			*37.9*			*4329.3*			
	Beam	8	0	9120	0	*0%*	19.3	0.00	*0%*	2165	0	*0%*	0	0	0
	Girder	8	0	9120	0	*standard*	18.6	0.00	*(standard)*	2165	0	*standard*	0	0	0
Hybrid Type I						*18804*			*86.0*			*4724.3*			
	Beam	4	4	4840	4840	*3%*	34.1	9.65	*127%*	1199	1209	*9%*	0	120	212
	Girder	4	4	4540	4584	*increase*	32.9	9.29	*increase*	1152	1165	*increase*	0	120	212
Hybrid Type II						*19148*			*40.5*			*4893.3*			
	Beam	4	4	4968	4840	*5%*	11.0	9.65	*7%*	1283	1209	*13%*	900	0	0
	Girder	4	4	4756	4584	*increase*	10.6	9.29	*increase*	1236	1165	*increase*	900	0	0

(L)= Components within the lateral system.
(G)= Additional components of the typical gravity sytem required for an equal comparison.

Connections

Individual Connections	Quantity	Plates Length (in) X	Plates Width (in) X	Plates Thickness (in)=	Plates Volume (in^3)	Plates Weight (lb)	Bolts Type	Bolts Diam. (in)	Bolts Quantity	Shop Weld Type	Shop Weld Throat	Shop Weld Passes	Shop Weld Length (in)	Eq. Length (in)
Typical Gravity connection														
Shear tab	1	18	5	0.375	33.8	**10**	A325	1	**6**	Fillet	5/16	1	36	**36**
Typical Moment Connection														
End Plate	1	36	8.5	1.25	382.5	109	A325	1	16	Full Pen.	9/16	2	65	131
Beam Stiffeners	2	6	10.39	0.75	46.8	13				Fillet	5/16	1	33	33
Doubler plate	1	21.58	26.76	0.5	288.7	82				Fillet	3/16	1	97	97
Transverse stiffeners	4	22.58	5	1.25	564.5	160				fillet	10/16	2	33	65
					Total=	**364**		Total=	**16**				Total=	**325**
Typical Hybrid Type I Connection														
Shear tab	1	18	5	0.375	33.8	**10**	A325	1	**6**	Fillet	5/16	1	36	**36**
Typical Hybrid Type II Connection														
Shear tab	1	12	4.5	0.375	20.3	**6**	A325	0.75	**4**	Fillet	0	1	24	**24**

System Comparison - Connections	Connection Count (L)	Connection Count (G)	Structural Steel Plates (lbs) (L)	Plates (lbs) (G)	Total (tons)		Bolting Quantity (L)	Quantity (G)	Total		Welding Eq. Length (in) (L)	Eq. Length (in) (G)	Total	
Moment Frame	96	0	34966	0	17.5	**0.0%**	1536	0	1536	**0.0%**	31236	0	31236	**0.0%**
						(standard)				**(standard)**				**(standard**
Hybrid Type I	48	48	460	460	0.5	**97.4%**	288	288	576	**62.5%**	1728	1728	3456	**88.9%**
						decrease				**decrease**				**decrease**
Hybrid Type II	48	48	276	460	0.4	**97.9%**	192	288	480	**68.8%**	1152	1728	2880	**90.8%**
						decrease				**decrease**				**decrease**

(L)= Components within the lateral system.
(G)= Additional components of the typical gravity sytem required for an equal comparison.

Foundations

Individual Component Volumes	**Length (ft) X**	**Width (ft) X**	**Depth (in)=**	**Volume**		**Diam. (in)**	**Length (ft)**	**Quantity**	**Vol. (in^3)**	**Weight**	
Typical Gravity Spot Footing:	15.5	15.5	30	*600.6*	*cf*	0.75	15	31	785	*2668*	*lb*
Typical Gravity Grade Beam:	14.5	1.5	30	*54.4*	*cf*	0.75	30	3	152	*516*	*lb*
Typical Moment Frame Spot Footing :	16	16	30	*640.0*	*cf*	0.75	15.5	32	837	*2846*	*lb*
Typical Moment Frame Grade Beam :	14	1.5	30	*52.5*	*cf*	0.75	30	3	152	*516*	*lb*
Typical Hybrid Masonry Type I Spot Footing:	16.5	16.5	30	*680.6*	*cf*	0.75	16	33	891	*3029*	*lb*
Typical Hybrid Masonry Type I Grade Beam:	13.5	10	30	*337.5*	*cf*	0.75	30	15	759	*2582*	*lb*
Typical Hybrid Masonry Type II Spot Footing:	12	12	30	*360.0*	*cf*	0.75	11.5	24	466	*1584*	*lb*
Typical Hybrid Masonry Type II Grade Beam :	18	5.5	30	*247.5*	*cf*	0.75	30	8	405	*1377*	*lb*

Foundation System Comparision			**Volume of Conc. (cf)**						**Reinforcement**					
	Bay Count		Columns		Grade beam				Columns		Grade beam			
	(L)	(G)	(L)	(G)	(L)	(G)	Total (cy)		(L)	(G)	(L)	(G)	Total (lb)	
Moment Frame	16	0	12800	0	840	0	505	**0.0%**	56916	0	8262	0	65178	**0.0%**
								(standard)						**(standard**
Hybrid Type I	8	8	10890	2402.5	2700	435	608	**20.4%**	48470	10672	20655	4131	83928	**28.8%**
								increase						**increase**
Hybrid Type II	8	8	5760	2402.5	1980	435	392	**-22.5%**	25337	10672	11016	4131	51156	**-21.5%**
								decrease						**decrease**
(L)= Components within the lateral system.														
(G)= Additional components of the typical gravity sytem required for an equal comparison.														